UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	Docket No. 50-443-LA-2
NEXTERA ENERGY SEABROOK, LLC	ASLBP No. 17-953-02-LA-BD01
(Seabrook Station, Unit 1)	

Hearing Exhibit

Exhibit Number: NER004

Exhibit Title: Testimony of NextEra Witnesses Said Bolourchi, Glenn Bell, and Matthew Sherman ("SGH Testimony")

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NEXTERA ENERGY SEABROOK, LLC	)	
	)	July 24, 2019
(Seabrook Station Unit 1)	)	

TESTIMONY OF NEXTERA WITNESSES SAID BOLOURCHI, GLENN BELL, AND MATTHEW SHERMAN

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TABLE OF ABBREVIATIONS

Individuals

GB	Glenn Bell
SB	Said Bolourchi
MS	Matthew Sherman

Shortened Names

ACI	American Concrete Institute
ACRS	Advisory Committee on Reactor Safeguards
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	ASTM International (formerly American Society for Testing and Materials)
C-10	C-10 Research & Education Foundation
EPRI	Electric Power Research Institute
FHWA	Federal Highway Administration
FSEL	Ferguson Structural Engineering Laboratory
ICRI	International Concrete Repair Institute
ISE	The Institution of Structural Engineers
JAB	John A. Blume & Associates
MA	Massachusetts
MIT	Massachusetts Institute of Technology
MPR	MPR Associates, Inc.
NBS	National Bureau of Standards Publications
NextEra	NextEra Energy Seabrook, LLC
NIST	National Institute of Standards and Technology
NRC	Nuclear Regulatory Commission
Seabrook	Seabrook Station Unit 1
SGH	Simpson Gumpertz & Heger Inc.
UK	United Kingdom
US	United States
USA	United States of America
VC Summer	Virgil C. Summer Nuclear Generating Station
Vogtle	Alvin W. Vogtle Electric Generating Plant

Acronyms

AAR	Alkali-Aggregate Reaction (or Reactivity)
ASR	Alkali-Silica Reaction (or Reactivity)
AZ	Azimuth
B.S.	Bachelor of Science
BSc	Bachelor of Science
CB	Containment Building
CCI	Combined Cracking Index
CEB	Containment Enclosure Building
CEO	Chief Executive Officer
CFR	Code of Federal Regulations
CI	Crack Index or Cracking Index
COLA	Combined Operating License Application
CSA	Canadian Standards Association
DRI	Damage Rating Index
E	Elastic Modulus or Young's Modulus (also see below under Loading)
FEM	Finite Element Method
GDC	General Design Criteria
LAR	License Amendment Request 16-03
LRA	License Renewal Application

LSTP	Large-Scale Test Program
M.S.	Master of Science
NQA	Nuclear Quality Assurance
Ph.D.	Doctor of Philosophy
PIC	Principal-In-Charge
PRA	Probabilistic Risk Assessment
SE	Safety Evaluation
SEM	Structural Evaluation Methodology
SMP	Structural Monitoring Program
SRP	Standard Review Plan
SSI	Soil-Structure Interaction
UFSAR	Updated Final Safety Analysis Report
2D	Two-dimensional
3D	Three-dimensional

Loading

D	Dead Load
E	Lateral Earth Pressure
E_{ss}	Safe Shutdown Earthquake
FL	Factored (Design) Load
L	Live Load
OBE	Operating Basis Earthquake
P_v	Pressure Variation
R_0	Normal Pipe Reaction
S_a	ASR Load
T_0	Normal Temperature
U	Ultimate Design Load
W	Wind Load
W_t	Tornado Wind

Units

ft	Foot
g	Gravity
Hz.	Hertz
in.	Inch
m	Meter
mm	Millimeter

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TESTIMONY OF NEXTERA WITNESSES SAID BOLOURCHI, GLENN BELL, AND MATTHEW SHERMAN

I. WITNESS BACKGROUND

A. Said Bolourchi ("SB")

Q1. Please state your full name.

A1. (SB) My name is Said Bolourchi.

Q2. By whom are you employed and what is your position?

A2. (SB) I am employed by Simpson Gumpertz & Heger Inc. ("SGH") as a Senior Principal. I am the Principal-In-Charge ("PIC") for all SGH projects associated with the evaluation of seismic Category I structures at NextEra Energy Seabrook, LLC's ("NextEra") Seabrook Station Unit 1 ("Seabrook") that are affected by Alkali-Silica Reaction ("ASR"). I am also the PIC for the SGH project supporting NextEra's License Amendment Request 16-03 ("LAR") and License Renewal Application ("LRA").

Q3. Please describe your role in this proceeding.

A3. (SB) I am involved in this proceeding as a NextEra witness in connection with the adjudication of the admitted contention. My role is to provide relevant information regarding the evaluation of ASR at Seabrook. More specifically, my role is to provide information on the

initial root cause analysis for structural deformation, on ASR monitoring at Seabrook, and on preparing SGH Document No. 170444-MD-01, Rev. 1, "Methodology for the Analysis of seismic Category I Structures with Concrete Affected by Alkali-Silica Reaction" (May 31, 2018) (INT022), also known as the Structural Evaluation Methodology ("SEM") Document. I have also been the PIC for evaluating all seismic Category I structures at Seabrook, which includes defining the long-term monitoring limits and frequency of monitoring for these structures, and the development of the LAR.

Q4. Please describe your educational and professional qualifications, including relevant professional activities.

A4. (SB) My professional and educational qualifications are summarized in my *curriculum vitae* (NER031). I have over 40 years of professional experience, primarily in the nuclear industry. By way of formal education, I earned my B.Sc. in Mechanical Engineering from Queen Mary College of London University in the UK, and my M.S. in Mechanical Engineering and Ph.D. in Applied Mechanics from Massachusetts Institute of Technology (MIT), Cambridge, MA. My Ph.D. thesis was on developing geometric and material nonlinear formulations of plate and shell elements for finite element analysis. I started supporting structural evaluation, upgrade, and licensing of the Diablo Canyon nuclear power plant when I joined John A. Blume & Associates ("JAB") in 1981. I was in charge of the containment gantry crane and Unit 2 Turbine building seismic analysis and upgrade for this plant. While I was at JAB (which was acquired by URS Corporation), I performed seismic analysis for many nuclear plants in the United States, including performing seismic soil-structure interaction analysis, developing in-structure response spectra, nonlinear seismic analysis for re-racking and condensed racking of fuel pools, and nonlinear seismic analysis of cable tie systems. I have also worked on natural phenomena hazard evaluations of test reactors and other nuclear processing

facilities for Department of Energy ("DOE") facilities. SGH became active in the nuclear field when I joined in 2002. I have been the SGH nuclear practice group lead since 2008. I was PIC for seismic analysis and design for four combined operating license applications ("COLA") for other seismic Category I structures at different sites. I have also been PIC for designing and performing laboratory testing of waterproofing membrane systems with a sufficient friction coefficient to meet the design requirements for the new Vogtle and V.C. Summer nuclear plants which are or were under construction. I was also PIC for heavy lift analysis and design of the stiffening members for the Vogtle and V.C. Summer nuclear plants. I was PIC for structural design and construction support for the high-hazard Integrated Waste Treatment Unit at the DOE Idaho site.

Q5. Are you familiar with the sections of the LAR (as defined in A36 of NextEra's Testimony (NER001)) that are relevant to the technical issues raised in the contention?

A5. (SB) Yes. I am familiar with the technical issues related to the impacts of ASR on reinforced concrete structures, the development and execution of analysis and evaluation of seismic Category I structures with concrete affected by ASR, and defining the monitoring frequency and ASR and structural monitoring parameters and limits. I have been involved with addressing ASR issues at Seabrook since NextEra first requested that SGH perform testing and petrography for some cores from Seabrook in 2010. I have knowledge of the large-scale test program ("LSTP") results and the consideration of those results applicable to the SEM. I also have personal knowledge of the evaluations of the Seabrook seismic Category I structures affected by ASR, the application of proposed structural monitoring parameters, and the frequency of monitoring based on structural evaluations under the SEM that become inputs to Seabrook's Structures Monitoring Program ("SMP").

Q6. Please further describe the basis for your familiarity with these aspects of the LAR and the associated technical issues raised in the contention.

A6. (SB) I prepared sections of the LAR related to the structural analysis, evaluation, and monitoring. I have been the SGH PIC for all Seabrook ASR projects since the original cores were sent to us for testing and petrography. I am familiar with the LSTP activities and provided some engineering support for one of the measurement techniques used as part of this program. I have reviewed MPR-4273, Rev. 0, "Seabrook Station - Implications of Large-Scale Test Program Results on Reinforced Concrete Affected by Alkali-Silica Reaction" (July 2016) (NRC008) (the non-proprietary ("NP") version) and (NRC009) (the proprietary ("P") version). I have also reviewed MPR-4273, Rev. 1, dated March 2018 ("MPR-4273") (INT019-R)(NP) and (INT021)(P). I have also reviewed MPR-4288, Rev. 0, "Seabrook Station: Impact of Alkali-Silica Reaction on Structural Design Evaluations" (July 2016) ("MPR-4288") (INT012)(NP); (INT014)(P), to understand the parameters and limits needed for the structural analysis and evaluations of the Seabrook seismic Category I structures. I have reviewed many publications on different technical approaches for analyzing the effects of ASR and constitutive formulations proposed by various authors. This information informed SGH's selection of an analytical technique for evaluation of ASR-affected structures to incorporate in the SEM. I have kept up-to-date on the impact of ASR on different nuclear structures and evaluations performed for these structures. I have also had meetings with Idaho National Laboratories regarding their computer software using ASR constitutive modeling that they were planning to use for analyzing the light water reactor life expectancy project. I am also aware of Electric Power Research Institute ("EPRI") material formulation and inspection procedures that EPRI recommended for use by utilities. I have also reviewed the issues raised in this contention and have sufficient experience

and technical knowledge in the field of analysis, structural evaluation and design, and field inspection to address these issues.

B. <u>Glenn Bell ("GB")</u>

Q7. Please state your full name.

A7. (GB) My name is Glenn R. Bell.

Q8. By whom are you employed and what is your position?

A8. (GB) I am employed by SGH as a Senior Principal. I am the primary supervisor for the determination of the load factors used in the SEM for the SGH projects involving evaluation of the Seabrook seismic Category I structures that are affected by ASR, including the analysis of the Containment Building.

Q9. Please describe your role in this proceeding.

A9. (GB) I am involved in this proceeding as a NextEra witness in connection with the adjudication of the admitted contention. My role is to provide relevant information regarding the evaluation of ASR at Seabrook. More specifically, my role is to provide information on the approach to the SEM, the determination of load factors, and the analysis of the Containment Building.

Q10. Please describe your educational and professional qualifications, including relevant professional activities.

A10. (GB) My professional and educational qualifications are summarized in my *curriculum vitae* (NER032). By way of formal education, I earned a B.S. degree in Civil Engineering from Tufts University and a M.S. degree in Structural Engineering and Structural Mechanics from the University of California at Berkeley. I have spent more than 44 years in practice, mostly at SGH. I was CEO of SGH from 1995 through 2016 and Chair of its Board of Directors from 2016 through 2018. And as SGH's CEO, I was responsible for the development

and implementation of, and compliance with, SGH's Quality Assurance for Nuclear Facilities ("QANF") program from 2002 through 2016.

My areas of expertise are in the design, evaluation, and rehabilitation of structures of all types. My experience includes the application of advanced analysis to the evaluation and failure analysis of complex structures. I have made contributions to national seismic code development and practices for forensic evaluation. I am a registered Professional Engineer in nine states and the District of Columbia, a registered Structural Engineer in Illinois, and a Chartered Engineer in the UK.

I have been involved in professional society and professional standards work for most of my career. I am currently President Elect of the US-based Structural Engineering Institute and a Board Trustee of the Institution of Structural Engineers in the UK. I have taught and lectured widely on various topics in structural engineering and was recently appointed the Galletly-Dickson Visiting Scholar at the University of Bath in the UK.

Q11. Are you familiar with the sections of the LAR (as defined in A36 of NextEra's Testimony (NER001)) that are relevant to the technical issues raised in the contention?

A11. (GB) Yes. I am familiar with the technical issues related to the impacts of ASR on reinforced concrete structures, evaluation of seismic Category I structures with concrete affected by ASR, and defining the measurements used for structural monitoring. I have been involved with addressing ASR at Seabrook since 2016. I have personal knowledge of the LSTP results and how they impact the SEM. I also have personal knowledge of the evaluations of the Seabrook seismic Category I structures affected by ASR and application of proposed structural monitoring parameters and frequency of monitoring included as inputs to Seabrook's SMP.

C. <u>Matthew Sherman ("MS")</u>

Q12. Please state your full name.

A12. (MS) My name is Matthew Sherman.

Q13. By whom are you employed and what is your position?

A13. (MS) I am employed by SGH as a Senior Principal. I am the primary supervisor for the field work and testing portions of the SGH projects associated with the structural evaluation of Seabrook structures affected by ASR.

Q14. Please describe your role in this proceeding.

A14. (MS) I am involved in this proceeding as a NextEra witness in connection with the adjudication of the admitted contention. My role is to provide relevant information regarding the evaluation of ASR at Seabrook. More specifically, my role is to provide information on the ASR mechanism, ASR monitoring at Seabrook, and evaluation of monitoring data.

Q15. Please describe your educational and professional qualifications, including relevant professional activities.

A15. (MS) My professional and educational qualifications are summarized in my *curriculum vitae* (NER033). By way of formal education, I earned my B.Sc. degree from Cornell University and my M.Sc. degree at the University of Texas at Austin. I have spent approximately 27 years in practice: 24 in consulting engineering and 3 working as a project engineer for a heavy civil contractor. My area of expertise is Civil/Structural Engineering, with a focus on construction materials, repair/rehabilitation, and testing. I am a Fellow of the American Concrete Institute ("ACI") and the International Concrete Repair Institute ("ICRI"). I presently serve on various ACI committees, including the Committee on Durability, Design of Nuclear Structures, Financial Activities, and Aggregates. Within ICRI, I serve on the Evaluation, Specifications, and Technical Activities committees. Technical papers that I

authored or co-authored have received various awards from ACI, including the Construction Award in 2015. I am a registered Professional Engineer (Civil) in New Hampshire (No. 13479) and ten other states.

Q16. Are you familiar with the sections of the LAR (as defined in A36 of NextEra's Testimony (NER001)) that are relevant to the technical issues raised in the contention?

A16. (MS) Yes. I am familiar with the technical issues related to the impacts of ASR on reinforced concrete structures, evaluation of seismic Category I structures with concrete affected by ASR, and defining the measurements used for structural monitoring. I have been involved with addressing ASR at Seabrook since SGH was first asked to perform testing and petrography for some cores from Seabrook in 2010. I have personal knowledge of the LSTP results and how they impact the SEM. I also have personal knowledge of the evaluations of the Seabrook seismic Category I structures affected by ASR and application of proposed structural monitoring parameters and frequency of monitoring included as inputs to Seabrook's SMP.

Q17. Please further describe the basis for your familiarity with these aspects of the LAR and the associated technical issues raised in the contention.

A17. (MS) I have been involved in multiple projects involving the practical implementation of ASR-related evaluations and monitoring, including developing an ASR management protocol for ASR-affected highway structures in Massachusetts, evaluating building foundations affected by ASR, and evaluating potential reuse of historic bridge foundations affected by ASR.

II. <u>GENERAL BACKGROUND</u>

Q18. Are you familiar with NextEra's LAR 16-03?

A18. (SB, GB, MS) Yes. We also note that a comprehensive summary of the LAR is provided in the Testimony of NextEra Witnesses Michael Collins, John Simons, Christopher

Bagley, Oguzhan Bayrak, and Edward Carley (NER001) ("MPR Testimony"). For the sake of brevity, we do not repeat it here in our testimony document.

Q19. Are you familiar with the regulations and guidance applicable to the LAR?

A19. (SB, GB, MS) Yes. We also note that a comprehensive summary of the applicable regulations and guidance is provided in the MPR Testimony (NER001). For the sake of brevity, we do not repeat it here in our testimony.

Q20. Are you familiar with the Technical Background discussion in the MPR Testimony?

A20. (SB, GB, MS) Yes. We have reviewed NextEra's summary of the applicable technical background as provided in the MPR Testimony (NER001). For the sake of brevity, we do not repeat it here in our testimony. However, our responses to various questions below provide additional detailed background regarding the SEM and other topics within the scope of our testimony.

Q21. Are you familiar with the Contention admitted by the Board for adjudication in this proceeding?

A21. (SB, GB, MS) Yes. The Board admitted for adjudication a single contention, reformulated as follows:

> The large-scale test program, undertaken for NextEra at the FSEL, has yielded data that are not "representative" of the progression of ASR at Seabrook. As a result, the proposed monitoring, acceptance criteria, and inspection intervals are not adequate.

The Board emphasized that the "key issue" is C-10's "challenge to the representativeness of the large-scale test program," and explained that the remaining aspects of the Contention merely assert "consequences" stemming from this alleged lack of representativeness.

Q22. What statements of position, testimony, and exhibits have you reviewed in preparation for the hearing?

A22. (SB, GB, MS) At this time, we have reviewed the following documents, filed by C-10, to the extent they are relevant to our testimony:

- C-10's Initial Statement of Position ("SOP") on C-10's Contentions Regarding NextEra's Program for Managing ASR at Seabrook Station Nuclear Power Plant, dated June 10, 2019, and Appendix A thereto (Exhibit List);
- Summary of Testimony of Victor E. Saouma, Ph. D., Regarding Scientific Evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction at the Seabrook Nuclear Power Plant, dated June 10, 2019 (INT002);
- Pre-filed Opening Testimony of Victor E. Saouma, Ph. D., Regarding Scientific Evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction at the Seabrook Nuclear Power Plant, dated June 10, 2019 (INT001-R); and
- C-10 Exhibits INT003 to INT027.

Q23. What other materials have you reviewed or do you expect to review in the preparation of your testimony?

A23. (SB, GB, MS) We have reviewed numerous documents in preparing this testimony, including, for example, those portions of NextEra's LAR relating to the SEM, and the pertinent portions of NRC regulations and guidance documents.

We will review the NRC Staff's pre-filed testimony, statement of position, and exhibits when those documents are filed, as well as any rebuttal filings by C-10.

Q24. Do you recognize Exhibit NER005?

A24. (SB, GB, MS) Yes. It is a list of NextEra's exhibits, and includes those documents which we referred to, used, or relied upon in preparing this testimony, including relevant codes, research papers, and guidance documents.

Q25. I show you Exhibits NER001, NER012 to NER013, NER028, and NER031 to NER044. Do you recognize these documents?

A25. (SB, GB, MS) Yes. These are true and accurate copies of the documents that we have referred to, used, and/or relied upon in preparing the respective parts of our testimony. In those cases in which we have attached only an excerpt of a document as an exhibit, that is noted on NextEra's exhibit list.

Q26. How do these documents relate to the work that you do as an expert in forming opinions such as those contained in this testimony?

A26. (SB, GB, MS) These documents represent the type of information that persons within our fields of expertise reasonably rely upon in forming opinions of the type offered in this testimony. Many are documents prepared by government agencies, peer-reviewed articles, or documents prepared by NextEra (or the utility industry). We note at the outset that we do not offer legal opinions on the NRC regulations or adjudicatory decisions discussed in our testimony. However, reading those regulations and decisions as technical statements, and using our expertise and experience, we interpret the meaning of those documents as they relate to how NextEra has addressed the issues raised in this contention. To the extent our testimony provides technical expert opinions on the requirements of NRC regulations, we believe that such opinions will be helpful to the Board inasmuch as they provide to the Board insights into NextEra's and the NRC Staff's processes for complying with the applicable regulations.

III. SUMMARY OF DIRECT TESTIMONY AND CONCLUSIONS

Q27. What is the purpose of your testimony?

A27. (SB, GB, MS) The purpose of our testimony is to demonstrate that the contention lacks merit, and, accordingly, should be resolved in NextEra's favor. In particular, we demonstrate that the LAR is based on a comprehensive review of available research and technical information. It provides reasonable assurance that the overall methodology for

addressing ASR-affected structures at Seabrook is satisfactory for ensuring that these structures can fulfill their design basis in accordance with applicable NRC regulations, guidance, and precedent. Specifically, the methodology in the LAR provides reasonable assurance that reinforced concrete structures at Seabrook will continue to meet the relevant requirements of 10 C.F.R. Part 50, Appendix A, GDC 1, 2, 4, 16, and 50 (Containment only for GDC 16 and 50), and 10 C.F.R. Part 50, Appendix B. In our professional opinions, C-10 fails to show that the LAR has any deficiency in this regard.

Q28. Please describe the scope of your testimony.

A28. (SB, GB, MS) Our testimony identifies and describes the pertinent portions of the LAR with a focus on the structural analysis and evaluation methodologies and the collection of field data for use in these evaluations. Our testimony provides an overview of the SEM, consistent with the Updated Final Safety Analysis Report ("UFSAR") as amended by the LAR, and relevant sections of NUREG-0800, the "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," Chapter 3, "Design of Structures, Components, Equipment, and Systems" ("SRP"). We show that the field data collection aspect of the SEM is fully consistent with concrete industry guidance for determining the degree and progression of ASR and using those results to support an analytical determination of the effect of that ASR, including:

- U.S. Department of Transportation, Federal Highway Administration, "Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures" (FHWA-HIF-09-004) (Jan. 2010) ("FHWA Guideline") (NER013);

- The Institution of Structural Engineers, "Structural Effects of Alkali-Silica Reaction" (July 1992) ("ISE Guideline") (NER012); and

- Canadian Standard Association International, "Guide to the Evaluation and Management of Concrete Structures Affected by Aggregate Reaction," General

Instruction No. 1, A864-00, (Feb. 2000) (Reaffirmed 2005) ("CSA Guideline") (NRC076).

We also show that the SEM, as implemented for the ASR-affected seismic Category I structure evaluations, meets the original design codes of record. Our testimony demonstrates that the SEM and its implementation for evaluating these structures provide reasonable assurance that these buildings meet design requirements.

Q29. Please provide an overview of your testimony.

A29. (SB, GB, MS) Our testimony opens with a background discussion of the SEM and its fundamental objectives, and goes on to discuss the details of the SEM, including: (1) ASR loads, load factors, and load combinations, (2) Seabrook field data, (3) finite element modeling, and (4) the use of this information, per the SEM, in the actual structural evaluations. Following our detailed discussion of the SEM, we directly address the specific claims related to structural evaluations included in the "Pre-filed Opening Testimony of Victor E. Saouma, Ph.D. Regarding Scientific Evaluation of NextEra's Aging Management Program for Alkali-Silica Reaction at the Seabrook Nuclear Power Plant, Submitted on Behalf of C-10 Research and Education Fund," dated June 10, 2019 ("Saouma Testimony") (INT001-R). To facilitate the review of our testimony, we have included a list of acronyms at the beginning of this document.

Q30. Please summarize Dr. Saouma's claims proffered in support of the Contention.

A30. (SB, GB, MS) As relevant to our Testimony, Dr. Saouma's key assertions (as summarized in the C-10 SOP), are that:

- ASR does not create "load" on structures (and therefore NextEra's method of evaluating structures by calculating ASR load is flawed);
- The LSTP was insufficiently representative of Seabrook, and therefore structural evaluations that use data from the LSTP as inputs into calculations are likewise flawed; and

- "[N]either NextEra nor NRC Staff obtained a peer review of their analyses of the ASR issue at Seabrook." C-10 SOP at 13.

Additionally, Dr. Saouma's Testimony (INT001-R) includes numerous comments purporting to challenge one particular document: the SGH "Evaluation and Design Confirmation of As-Deformed [Containment Enclosure Building or ("CEB")], 150252-CA-02," Rev, 0, (July 2016) ("Rev. 0 CEB Evaluation") (INT015), which was submitted as Enclosure 2 to Letter SBK-L-16153, re: Seabrook Station (Sept. 30, 2016).

Q31. Do you disagree with Dr. Saouma's and C-10's claims as set forth in the Contention?

A31. (SB, GB, MS) Yes. Our testimony demonstrates that these claims lack merit.

Q32. Please summarize the bases for your disagreement with Dr. Saouma's proffered claims.

A32. (SB, GB, MS) As we fully explain throughout this testimony, and as the NRC Staff concluded in its Final Safety Evaluation (Mar. 11, 2019) ("Final SE") (INT024)(NP), (INT025)(P), NextEra's LAR fully complies with all legal and regulatory requirements. The following summarizes the principal bases for our disagreement with C-10 and Dr. Saouma's key claims:

- Contrary to Dr. Saouma's opinions, *actual observations at Seabrook* confirm that ASR does create "load" on structures.
- Contrary to Dr. Saouma's assertion, no data from the LSTP are used as inputs into the structural evaluations performed under the SEM.
- Contrary to C-10's claim, the LAR and its component parts underwent independent review at multiple levels.
- The Rev. 0 CEB Evaluation has been superseded, and Dr. Saouma's criticisms are out of date and otherwise meritless.

Thus, as the NRC Staff concluded in its Final SE, NextEra's LAR demonstrates reasonable assurance and fully complies with all legal and regulatory requirements. *See* Final SE at 64 (INT024)(NP), (INT025)(P).

IV. STRUCTURAL EVALUATION METHODOLOGY (SEM)

A. Overview

Q33. What is the purpose of the SEM?

A33. (SB, GB, MS) The purpose of the SEM is to provide a method for analyzing and evaluating seismic Category I structures with concrete affected by ASR. The detailed analysis and evaluation procedures permit all seismic Category I structures to be evaluated using the same methodology and provide clear guidance to engineers performing the structural evaluation with sufficient procedural details to be repeatable by other knowledgeable engineers.

Q34. Does the LAR propose an entirely new method for evaluating structural adequacy?

A34. (SB, GB, MS) No. Our goal was to develop a methodology to evaluate whether a given structure affected by ASR meets the intent of the *original design criteria, standards, and codes of record* and achieves the structural safety reliability indices consistent with the original design. Because the original codes do not provide a means to account for the effect of ASR, our approach was to develop a "backfit" onto Seabrook's existing codes, and to do so in a way that maintains the level of expected structural performance implicit in the original design criteria, codes, and standards.

Q35. Is the SEM designed to predict the final growth of ASR in a given structure?

A35. (SB, GB, MS) No. Consistent with the original design codes of record, the methodology is designed to establish certain limits (akin to acceptance criteria) for future ASR expansion within which the structure can be deemed structurally adequate.

Q36. What do you mean by the original design criteria, codes, and standards?

A36. (SB, GB) This refers to the structural criteria, including referenced codes, standards, and specifications in the UFSAR for the original plant design. The referenced codes for Seabrook are the ASME Boiler and Pressure Vessel Code Section III, Div. 2, 1975 ("ASME 1975") (NRC050) for the Containment Building, and ACI Standard 318-71 ("ACI 318-71) (NRC049) for Seabrook's other seismic Category I structures.

Q37. In basic terms, how do the original design codes of record evaluate structural adequacy?

A37. (SB, GB) Evaluations of structural adequacy are exercises to determine whether the "demands" (i.e., load effects) on a structure or its elements exceed the "capacities" (e.g., strength or stress limits) of the structure or its elements. Methods of determining appropriate demands and capacities are prescribed by specific criteria, standards, and codes. For Seabrook, these methods are described in its UFSAR at Section 3.8.

Q38. Does ASR necessarily degrade the *capacity* of concrete?

A38. (SB, GB, MS) Not always. As reported in the technical literature such as the ISE Guideline (NER012) and observed in the LSTP as documented in MPR-4273 (INT019-R) (NP), (INT021) (P), one effect of ASR is that material properties of *unconfined* concrete are reduced. Specifically, its compressive strength exhibits a minor decrease as a function of ASR progression, while elastic modulus and tensile strength are much more sensitive. However, for *reinforced* concrete structures, this decrease in the *unconfined* material properties may not result in a decrease in structural performance due to the chemical prestressing effect. For example, the ISE Guideline at 14 (§ 4.4) (NER012) notes that "[i]t is emphasized that the residual strengths and stiffnesses in actual structures will be modified from [those of freely expanding concrete] because the concrete in actual structures is generally restrained by adjacent material…." and

SOMERVILLE, G., "MANAGEMENT OF DETERIORATING CONCRETE STRUCTURES" (2008) ("Somerville") provides modifications and correction values that are to be used when evaluating concrete in which the expansion is restrained. For the limit states of shear and flexure, results from laboratory testing suggest that the effects of confinement from the reinforcement more than compensate for degradation of material properties when ASR progression is within the range observed in laboratory testing. *See* MPR-4288 (INT012)(NP), (INT014)(P).

Q39. Generally speaking, how does the SEM treat the *stiffness* and *capacity* of ASR-affected reinforced concrete structures?

A39. (SB, GB, MS) Because laboratory testing (the LSTP) demonstrated that stiffnesses and capacities of reinforced concrete structural members were not compromised when expansion caused by ASR was within the range observed in laboratory testing (i.e., the limits of the testing), and NextEra has incorporated those limits in its SMP (i.e., the "Expansion Monitoring Limits," as further discussed in the MPR Testimony (NER001)), the SEM computes the elastic modulus of concrete using the minimum specified design compressive strength from the original design criteria (i.e., with no reduction for ASR). The remaining material properties used in the calculations also are consistent with those in the original design calculations.

Q40. Other than reviewing the LSTP results regarding the effect of ASR on the *stiffness* and *capacity* of reinforced concrete, was any other data from the LSTP used as an input into the SEM or the structural evaluations for specific structures at Seabrook?

A40. (SB, GB, MS) No. No specific measurements, calculations, data, or other information from the LSTP are direct inputs into the SEM or structural evaluations; and nothing from the LSTP informed any baseline assumptions on the *demand* side of the equations for the SEM or the structural evaluations. We evaluated the LSTP information regarding stiffness and capacity in much the same way that we evaluated other academic literature and testing programs. We considered these conclusions (i.e., that stiffness and capacity are not impacted by ASR

within the limits of testing) in developing the baseline assumption in the SEM that existing code capacities and standard methods of computing stiffness can be used in structural analyses and evaluations. Additionally, we performed our *own* analytical study to evaluate the impact of ASR on stiffness and capacity, as documented in NextEra's Response to RAI-D10 (NRC015). *See* Simpson Gumpertz & Heger calculations supporting the response provided to RAI D10 regarding cracked section properties used for the evaluation of the RHR vault and Spent Fuel pool walls (Enclosure 5 to SBK-L-018074 (June 7, 2018)) (INT023).

Q41. Based on your review of Dr. Saouma's testimony, does he dispute your assertion that reinforced concrete structures do not always show a decrease in structural performance (i.e., *capacity*) due to ASR?

A41. (SB, GB, MS) No. In fact, Dr. Saouma admits that "[m]any tests have shown an *increase* in structural shear strength in reinforced concrete beams (through the so-called prestressing effect) because of ASR." Saouma Testimony at 6 (INT001-R) (emphasis added).

Q42. Does ASR create *demand* on concrete?

A42. (SB, GB, MS) Yes. As explained further below in A52 through A54, concrete expansion caused by ASR can produce loads (i.e., demand) on concrete structures if the concrete is restrained by: (1) reinforcement, (2) other elements of the structure, or (3) external restraints on the structure (e.g., surrounding bedrock). Thus, the SEM was intended to assure that the original design capacities were not exceeded by updated demands due to ASR expansion. We also note that, whereas MPR's work (including the LSTP) supports the methodology for the *capacity* side of the code equations, ASR *demand* and the *comparison* of capacity to demand are the focus of SGH's contribution to the LAR.

Q43. What are the key features of the "backfit" onto the codes to account for ASR loads (i.e., *demand*) on concrete structures?

A43. (SB, GB, MS) The SEM provides a methodology for calculating the load imposed on the structure by the ASR itself. The calculated ASR load gets "plugged-in" to the code equations along with the other existing loads provided in the UFSAR to calculate the structural adequacy.

Q44. Can you provide a brief overview of the development of the ASR loads?

A44. (SB, GB, MS) Yes. As explained in more detail below in Section IV.C, field measurements of ASR expansion of Seabrook structures (not the LSTP) are used to calculate ASR loads. When the evaluation of a structure indicates that remaining design margin exists, ASR loads are amplified by a "threshold factor" such that the controlling demands on the structure are equal to (or slightly less than) the capacity of the structure.

Q45. Can you please briefly elaborate on the "threshold factor" concept?

A45. (SB, GB, MS) Yes. The threshold factor represents the reserve design margin in the structure for accommodating increasing future ASR demands. In simplified terms, it is an output of the structural evaluation calculation that represents the *remaining margin to the code limit*, explained in more detail below in A81. Once the threshold factor is established, the structure is monitored, and quantitative measurements and qualitative observations are compared to the specified limits defined by structural evaluation calculation. If the measurements or observations begin to approach the specified limits, further structural evaluations can be performed per the SEM.

Q46. To summarize your answers above, in the simplest possible terms, how does the SEM determine structural capacity and demand?

A46. (SB, GB, MS) Capacities are determined using the existing design codes and specifications prescribed in UFSAR Section 3.8. Demands are determined using: (1) the original

loads, load factors, and load combinations prescribed by the UFSAR (more fully described in A51 through A59), and (2) adding the *new* loads representing the load effects of ASR.

Q47. At a high-level, how does the SEM use capacity and demand to analyze seismic Category I structures at Seabrook?

A47. (SB, GB, MS) A three-stage analysis approach is used for analyzing and evaluating Seabrook's seismic Category I structures. Higher stages apply more sophisticated methods and use additional field measurement data of ASR expansion to improve the rigor of the analysis. The analysis and evaluation of each structure may begin at any stage and, if necessary, progress to a higher analysis stage. The three analysis stages are described in further detail below in A50. Stage Two and Stage Three evaluations use finite element modeling, which is described in further detail below in Section IV.D.

Q48. You stated that field data from Seabrook are used in both (a) calculating ASR loads and (b) performing structural evaluations. Can you briefly describe these data?

A48. (SB, GB, MS) Yes. Field data can include observations (non-quantifiable information), measurements (quantifiable information and data used directly in analysis), and specialty testing (supporting or interpretive information and data). The collection and use of field data are explained in further detail below in Section IV.C.

Q49. Please summarize the analytical approach that is used in the SEM to perform a structural evaluation for ASR-affected structures.

A49. (SB, GB) The SEM assesses the impact of self-straining loads, including ASR, on the safety and reliability of Seabrook structures. The original design loads specified in the UFSAR are combined with the self-straining loads from ASR expansion, creep, shrinkage, and swelling (see Section IV.B, below, for a further discussion of "loads"). The structural demands, including the impact of self-straining loads, are compared to the code-defined capacities; and deformations from adjacent structures are compared to field-measured seismic gaps.

The methodology also considers the impact of expansion of the concrete used to backfill the space between the Seabrook structures and the surrounding rock, as illustrated in Figure 1 below. Because the backfill concrete used during the original construction used the same materials and proportions as the structural concrete, it is also susceptible to slowly-occurring ASR.

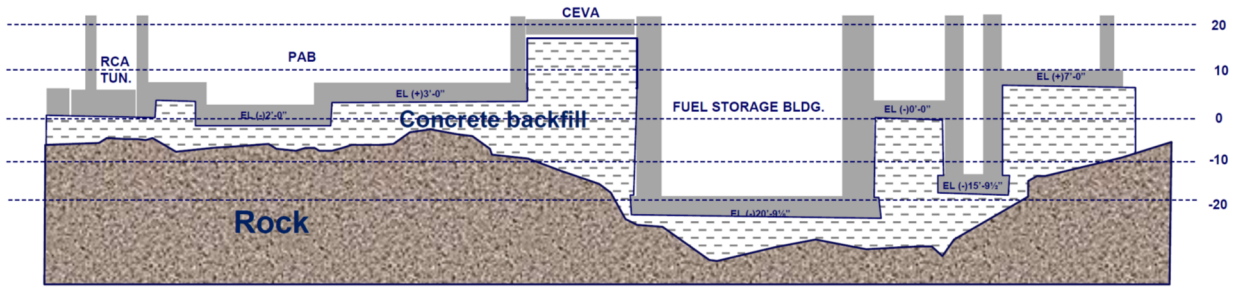

Figure 1 - Illustration of bedrock, concrete backfill, and plant structures.

Q50. Please describe the SEM's overall approach for evaluating structures.

A50. (SB, GB) Fundamentally, the SEM uses a stepwise approach to evaluation, increasing the complexity and detail of the evaluation depending on the findings at each step, much in the same way as the FHWA Guideline (NER013) approach. The SEM uses a three-stage analysis methodology; the stage of analysis employed depends upon various factors, including margin in the original design (i.e., degree to which the original design strength may have exceeded the requirements of the UFSAR), magnitude of ASR observed, and observed distress level for each structure. Higher stages of the analysis apply more sophisticated methods and use additional field data where refinement is required or desirable.

- Stage One uses few field data and simple but conservative calculations to determine ASR loads to be added to the original design loads, with appropriate load factors, to evaluate structural adequacy. The use of finite element analysis is optional for Stage One.
- Stage Two uses more complex finite element analyses for a more precise analytical evaluation of ASR load demands to be added to the original design loads to evaluate total demand on the structure. These total demands are then compared to the original code capacities.

- Stage Three uses finite element analyses for calculating the total demands due to original design loads as well as self-straining loads including ASR, creep, and moisture swelling. The Stage Three analysis requires additional field inspections and more effort to validate the finite element model. The validation includes corroborating the structural deformation and stresses calculated by finite element analysis subjected to in-situ load conditions with field-measured deformation and observed conditions. The field measurements consider structural deformation (such as bowing of a wall or slab), and relative deformation between adjacent structures (such as seismic gap measurements, or annulus measurement between the CEB and the Containment Building). Observations include signs of structural distress (such as flexural cracks, which are parallel to supports, or shear cracks, which are diagonal to the supports).

The acceptance criteria for the Containment Building are found in ASME 1975 (NRC050). The acceptance criteria for all other seismic Category I structures are found in ACI 318-71 (NRC049).

B. ASR Loads, Load Factors, and Load Combinations

Q51. Can you please explain what the term "load" means as used throughout this testimony?

A51. (GB) As defined by ASCE/SEI, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures 7-16" ¶ 1.2.1 (2017) ("ASCE/SEI 7-16") (NER034), "Loads" are:

> Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movements, and restrained dimensional changes. Permanent loads are loads in which variations over time are rare or of small magnitude. All other loads are variable loads.

There are many types of loads that are relevant to the Seabrook structures. Examples are dead loads (the fixed weight of the structure), live loads (such as the time-varying weight of contents, e.g., temporary storage of materials), wind, earthquake effects, and temperature effects. All of the relevant original loads for Seabrook are contained in the original UFSAR. As mentioned above, the SEM provides for the addition of ASR loads to the original design-basis loads.

Q52. Can ASR cause a load on structures?

A52. (SB, GB) Yes. Concrete expansion caused by ASR will produce an internal or external load depending on whether the member has internal or external restraint. ASCE/SEI 7-16 defines forces or other actions that result from restrained dimensional changes as "Load." *See* ASCE/SEI 7-16 ¶ 1.2.1 (NER034). If ASR expansion is restrained, it will cause loads to develop as the restraint "pushes back" against the expansion. *See id.*

Q53. How does ASR produce load?

A53. (SB) The chemical reaction of ASR produces an ASR gel that causes concrete to tend to expand, elongating the affected member. If there is no internal or external restraint, the concrete ASR expansion will cause the member to expand freely, and no loads are developed. However, if the concrete is restrained internally (such as by reinforcing steel), the restraint will create *compressive* load in the concrete and *tension* in the reinforcement. When the concrete affected by ASR is restrained externally (for example, by external boundary conditions such as rock or other supports, or other structural elements), compressive load will be generated in the ASR-expanded concrete and in the external restraining element.

Q54. Please further describe the phenomenon of internal and external loads.

A54. (SB) Consider an unreinforced concrete beam expanding in one direction along its length due to ASR. (ASR expansion is volumetric but, for understanding and visualizing the load-generation process, a one-dimensional simplifying assumption is useful). As Figure 2(a) shows schematically, the concrete beam length increases without any internal or external loads.

Figure 2(b) schematically presents the behavior of the same ASR-affected beam but with reinforcement. The beam length increases but less than in the unreinforced beam. The concrete expansion is resisted by reinforcement, which sets the reinforcement in tension and concrete in compression (so-called chemical prestressing behavior). This is called an internal force, since

the loads between the concrete and steel are balanced, and there is no net force on the beam section.

If the original beam or beam with reinforcement has external restraint, the ASR expansion will be resisted at the restraining locations (Figure 2(c) schematically shows the case of a reinforced beam). The external restraint produces a net force on the beam, which will be balanced by forces at restraint locations. The ASR expansion also produces internal forces in a reinforced beam, as discussed above.

In summary, the ASR expansion will cause internal forces for reinforced concrete members and/or external load for restrained reinforced or unreinforced concrete members.

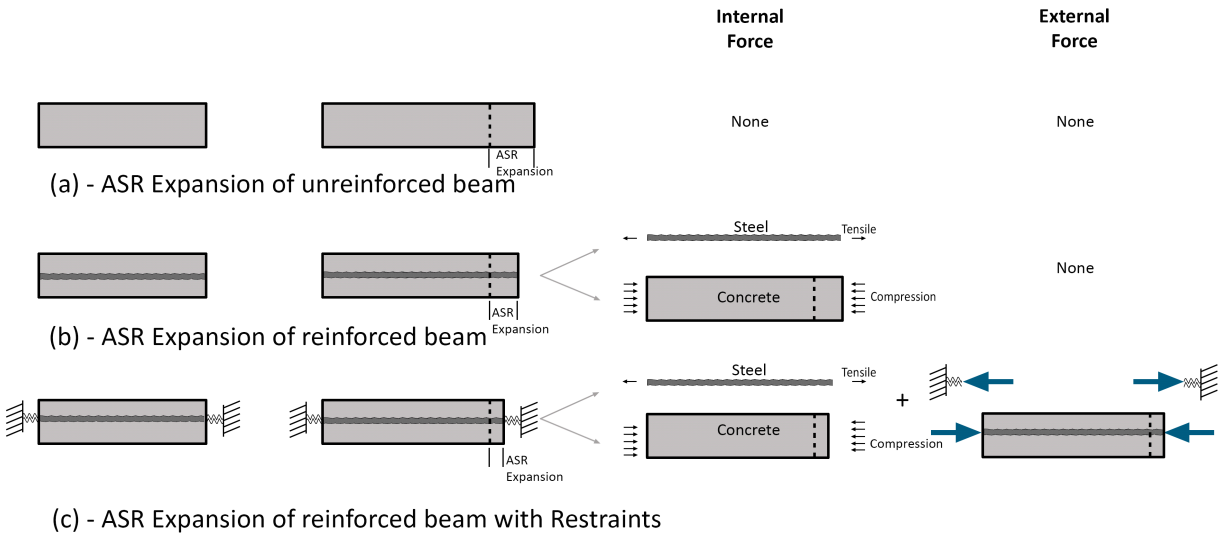

Figure 2 - Schematic Expansion of a Concrete Beam Affected by ASR

Q55. What are load factors?

A55. (GB, SB) Designers apply loads in the generic form "α(P)" where "P" is the load and "α" is the "load factor." In other words, load factors are "multipliers" to design loads. Load factors account for the likelihood that actual load on a structure may vary from (in particular, may be higher than) the nominal design loads used in calculations of structural adequacy. Their purpose is to add a measure of conservatism or factor of safety when there is a reasonably

foreseeable probability that the nominal design loads may be exceeded during the structure's life. Generally, load factors are prescribed by project-specific criteria, standards, and codes, as they were in the case of Seabrook through the UFSAR based on the SRP and the two relevant codes and standards: ASME 1975 (NRC050) and ACI 318-71 (NRC049).

Load factors are generally different for various types of loads, such as dead load, live load, wind load, earthquake load, etc. Load factors also depend on how the design loads are determined and what scenarios they represent. For example, if a design load represents a frequently-occurring scenario, then generally higher load factors must be applied to assure strength under more extreme scenarios. If the design load represents a scenario that is the maximum that is reasonably foreseeable for the structure, then a load factor of 1.0 is typically used. A load factor of 1.0 means that a load higher than the design load is not reasonably foreseeable in the design scenario; since the load itself includes an appropriate degree of conservatism, further increase is not warranted. As in the case of ASME 1975 (used for the Containment Building) and ACI 318-71 (used for Seabrook's other seismic Category I structures), different codes may use different approaches to load factors.

Q56. What are load combinations?

A56. (GB, SB) *Individual* loads on structures (dead, live, wind, earthquake, etc.) typically do not occur in isolation. Load *combinations* recognize the probability of two or more loads occurring simultaneously. The original UFSAR prescribed the various combinations of loads (with their associated load factors) that must be considered for the Containment Building and for Seabrook's other seismic Category I structures.

For example, one of the load combinations prescribed in the UFSAR Markup Table 3.8-16 for Seabrook's other seismic Category I structures (besides the Containment Building) is:

$$U = 1.4D + 1.7L + 1.7W + 1.7E + 2.0S_a$$

Where:

U = the so-called "Ultimate" design load = the structure's required strength
D = Dead Load (and any hydrostatic load applied to the structure)
L= Live Load
W=Wind
E=Lateral Earth Pressure
S_a=ASR Load

See NextEra Energy Seabrook's Evaluation of the Proposed Change Including Attachment 1 Markup of UFSAR Pages, tbl. 3.8-16 ("UFSAR Markup") (Aug. 1, 2016) (INT010) (NP); (INT011) (P); (NRC089).

The coefficients in front of D, L, W, and E are the load factors. Again, the SEM added ASR loads with associated load factors (i.e., $2.0S_a$) to the original load combinations.

Q57. How were the load factors determined for use in the SEM?

A57. (GB, SB) The SEM employs all of the original load factors and load combinations of the original design specified in the UFSAR, supplemented by load factors to account for the added ASR loading. *See* SGH Report 160268-R-01, Rev. 0, "Development of ASR Load Factors for Seismic Category I Structures (Including Containment) at Seabrook Station, Seabrook, NH (July 2016) ("SGH Load Factors Report") (INT013) documents the specific procedures for the Containment Building, based on ASME 1975 (NRC050), and for the seismic Category I structures other than the Containment Building, based on ACI 318-71 (NRC049).

Q58. Please summarize how the load factors were determined for the Containment Building using ASME 1975.

A58. (GB) ASME 1975 uses a scenario-based deterministic approach. In this approach, extreme values (i.e., maximum foreseeable values within a postulated scenario) are used, and therefore the load factors typically are 1.0. *See* ASME 1975 (NRC050). In the UFSAR, there are some load factors greater than 1.0 (e.g., 1.25 to 1.5) for severe environmental or abnormal conditions. An example is that the load factor applied to the Operating Basis Earthquake ("OBE") (which is a relatively frequently-occurring earthquake) is factored by 1.25 or 1.5 to represent certain extreme or abnormal scenarios, which are posited to occur very rarely in the life of the structure. That is, if a particular design load is frequently occurring over the life of the structure, it must be "scaled up" with a load factor greater than 1.0 for use in load combinations that are intended to represent extreme or abnormal scenarios.

The approach used in the SEM for ASR load factors for use with the ASME code was to posit a conservative ASR load scenario that has a very small likelihood of exceedance *in the present condition* and to use a load factor of 1.0. ("In the present condition" refers to the fact that the structural analysis employs loads that represent the current state of ASR. As stated above, the purpose of the SEM is not to predict future ASR expansion. Controls over the consequences of future ASR expansion are handled through the threshold factor (*see* A81, A82) and the Expansion Monitoring Limits in the SMP.

The ASR loads for the Containment Building were determined by defining four zones of ASR severity—Zones 1 through 4—with Zone 4 enveloping the highest severity observed to date at any of the Seabrook seismic Category I structures. Each zone has a lower and upper limit of ASR severity, as measured by the Cracking Index ("CI") (which is further explained in A63). There is also a Zone 0 which is defined as an area that has no indication of ASR. At each part of

a structural surface that exhibits ASR, the SEM maps one of the four zones onto it such that the actual ASR CI measurements fall within the lower and upper limits of that zone. (The SEM similarly uses four zones for all of Seabrook's other seismic Category I structures designed and evaluated by ACI 318-71).

As a conservative initial screening for the Containment Building, the SEM determines an ASR load for each zone that represents the CI upper limit of each zone conservatively increased by 25%. The 25% increase is to add a measure of conservatism to ensure no ASR within that zone is above that recorded or observed. At each zone, the augmented ASR design load is employed across the entire zone. Load factors are applied to all loads, including ASR, in the various load combinations. (Note that this approach is mathematically equivalent to using the unmodified upper limit of CI in each zone as a load in combination with a load factor of 1.25, but the approach of applying a more extreme load in combination with a load factor of 1.0 is more philosophically in keeping with the ASME 1975 scenario-based approach).

For example, one of the load combinations prescribed in the UFSAR Markup (Table 3.8-1) for the Containment Building is:

$$FL = 1.0D + 1.0L + 1.0T_0 + 1.0E_{ss} + 1.0W_t + 1.0R_0 + 1.0P_v + 1.0S_a$$

Where:

FL = Factored (Design) Load
D = Dead Load
L = Live Load
T_0 = Normal Temperature
E_{ss} = Safe Shutdown Earthquake
W_t = Tornado Wind
R_0 = Normal Pipe Reaction
P_v = Pressure Variation
S_a = ASR Load

See UFSAR Markup tbl. 3.8-1 (INT010) (NP); (INT011) (P); (NRC089).

Notably, the Containment Building displays only limited areas of low levels of ASR (i.e., within Zone 1 severity).

Q59. Please summarize how the load factors were determined for seismic Category I structures other than Containment using ACI 318-71.

A59. (GB, SB) Determination of the load factors for seismic Category I structures other than the Containment Building employed a different methodology than for the Containment Building because ACI 318-71 uses a different approach to loads and performance criteria than used by ASME 1975. Whereas, as described above, ASME 1975 uses scenario-based upper-bound loads coupled with acceptability criteria that limit stresses, ACI 318-71 is based on design loads that represent more commonly occurring, generally mean value, events; and acceptability is judged by development of adequate ultimate strength, which tolerates more extreme behavior than the stress limitations of ASME 1975. *See* ACI 318-71 (NRC049). (The performance limits on the Containment Building are more restrictive than the Seabrook's other seismic Category I structures because the Containment Building serves the special purpose of containing a hypothetical radiological release). Because ACI 318-71 is based on mean (average) loads within the realm of probability of those loads and ASME is based on more extreme (upper-bound) loads, the ACI 318-71 approach generally requires higher load factors than the ASME 1975 approach to achieve an appropriate level of reliability.

For the ACI approach, the SEM directly uses probabilistic insights to develop load factors. Since the 1980s, the underlying approach to the design of concrete structures to ACI 318 has been based on probabilistic concepts of structural reliability (i.e., the probability of not exceeding the acceptable measure of structural performance).

While the 1971 version of ACI 318 (i.e., ACI 318-71) was not based on probabilistic concepts, work was undertaken in the mid to late 1970s to define a probabilistic approach for

versions of ACI 318 following the 1971 version. The key project in laying the ground work for, and defining the principles of, probabilistically-based structural reliability was undertaken at the National Bureau of Standards ("NBS") (now the National Institute of Standards and Technology, or "NIST"). The leader of that program was Dr. Bruce Ellingwood, who was then a research engineer at the NBS. In their work, Dr. Ellingwood and his colleagues were able to quantify the levels of reliability inherent in then-existing design methods, including ACI 318-71 (and other standards), and they developed an approach for determining load factors going forward based on the levels of reliability inherent in prior standards. The work of Ellingwood and his colleagues culminated in a the following report: U.S. Department of Commerce, National Bureau of Standards, NBS Special Publication 577, "Development of a Probability Based Load Criterion for American National Standard A58" (1980) ("NBS Publication 577") (NER035). This report became the seminal document in structural reliability implementation in codes and standards and is the key reference in our approach to developing ASR load factors for Seabrook's other seismic Category I structures, which were based on the ACI Code. Under this approach, structural reliability is quantified by a reliability index, β. There is a relationship between probability of failure and β; the higher β, the higher the probability that the structure will remain safe.

For seismic Category I structures other than the Containment Building, the methodology to incorporate ASR effects into the analysis employs the NBS study findings regarding the reliability inherent in ACI 318-71. Similar to the method mentioned above for the Containment Building, for Seabrook's other seismic Category I structures, we divided the severity of ASR in the structure into four zones. For each zone, we determined ASR strains from CI statistics gathered over those zones. The calculations employed CI data from over twenty buildings at Seabrook in total, including 216 CI measurements on 108 grids. Using the statistics of CI

measurements in each severity zone, we were able to compute ASR load factors for each zone that maintained the targeted levels of reliability from NBS Publication 577 (NER035). In other words, Ellingwood's work defined the reliability levels inherent in the original codes and standards, and we computed the load factors for ASR loads that would result in an equivalent or higher level of reliability.

While this method is not, strictly speaking, a probabilistic risk assessment ("PRA"), it does account probabilistically for the variation in ASR across the plant in a manner that employs the same procedures and maintains the same reliability inherent in Seabrook's original design basis.

Q60. Was this aspect of the SEM independently reviewed?

A60. (SB, GB) Yes. Dr. Ellingwood, one of the authors of NBS Publication 577 (NER035) and an eminent authority on the theory of structural safety and reliability, reviewed and validated our work in this area. Dr. Ellingwood concluded as follows:

> ln my opinion, the methods employed . . . for revising the load combinations in the UFSAR for the Seabrook Station for ASR demands on Category I structures are, in general, consistent with the state of the art of structural reliability assessment and the development of probability-based load and resistance factors for structural design. Furthermore, the methods employed for revising the load combinations for the Containment . . . are entirely consistent with the conservative deterministic approach to safety assurance historically taken in developing the ASME Code.

SGH Load Factors Report, Attachment 1 (INT013). Dr. Ellingwood also testified regarding his review and validation before the Advisory Committee on Reactor Safeguards ("ACRS"), which also conducted an independent review of our work and determined that it was sound.[1] Moreover,

[1] *See* Transcript, Advisory Committee on Reactor Safeguards, License Renewal Subcommittee at 101-02 (Oct. 31, 2018) (ML18348B117).

experts at Brookhaven National Laboratory supported the NRC Staff's review of this methodology and agreed with its validity.[2]

C. Use of Field Data

Q61. How are the current levels of ASR expansion and deformations of the structure established?

A61. (SB, MS) As described in the FHWA Guideline, an "in-situ investigation program which includes monitoring of expansion and deformation generally provides the most reliable 'prognostic' for ASR-affected structural members." FWHA Guideline at 4 (NER013). At Seabrook, this is accomplished through a formal program that includes: walkdown; CI measurements and CCI determination; measurement of structural deformations between structures, seismic gaps, out-of-planarity for walls and slabs, and structural relative movement; measurement of pin-to-pin in-plane expansion; and determination of structural cracking. These are used to establish the current ASR level of expansion and deformations of the structure.

Q62. Please explain the Walkdown aspect of this program.

A62. (SB, MS) For each structure, a walkdown inspection is first performed to understand field conditions and to determine whether ASR expansion only affects localized regions of the structure or whether the structure has experienced global deformations. The walkdown inspection primarily includes qualitative observation with limited measurements to provide initial assessments of the severity and location of cracking. These observations are not used to generate data to be used directly in the analysis, but instead indicate if formal measurements are required as follow-up work. They also are used to guide the selection of any measuring points by identifying the areas of most prominent cracking and the overall structural configuration.

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[2] *See id*. at 229.

Q63. Can you please explain how the Cracking Index works?

A63. (MS, SB) Yes. The CI is a quantitative measurement of ASR-induced in-plane strain obtained by crack width summation and normalization. The measurement methodology is based on the methodology in the FHWA Guideline (NER013), CSA Guideline (NRC076), and ISE Guideline (NER012). The measurement is obtained via optical measurement accurate to approximately two one-thousandths of an inch. Figure 3 below shows an example of how a measurement is collected; Figure 4 shows a typical optical measurement device.

Figure 3

Figure 4

CI includes measurement and summation of crack widths along a set of perpendicular reference lines on the surface of a concrete element being investigated. The sum of crack widths is normalized by the length of the reference lines to determine the CI in-plane expansion. The CI measurement captures surface crack widths from *all sources of cracking* and thus can be

influenced (i.e., increased) by cracks from phenomena *other than from ASR*. A typical grid with width measurements is shown in Figure 5 below.

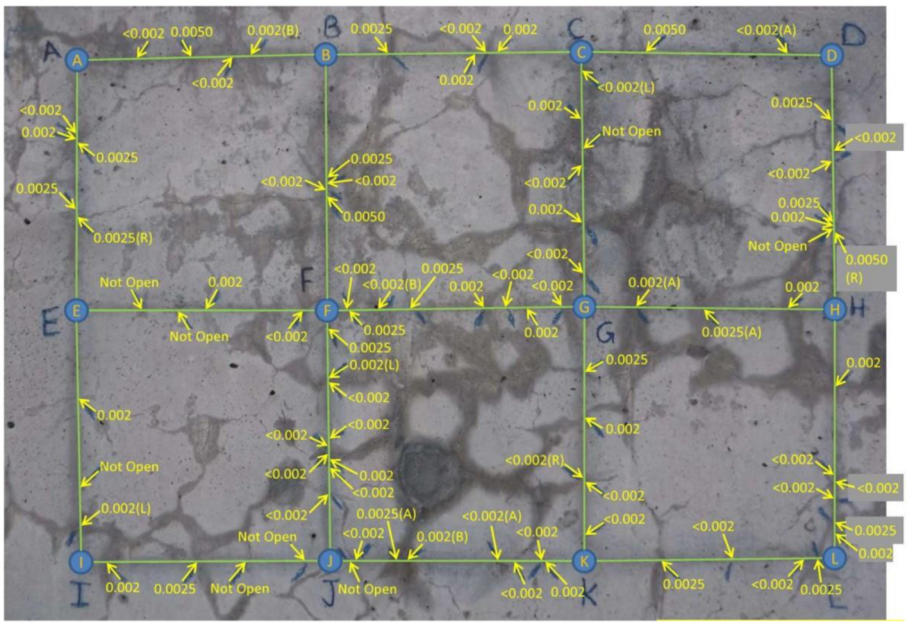

Figure 5

The Combined Cracking Index ("CCI") is the weighted average of the CI in the two measured in-plane directions. A typical ASR-monitoring location produces two CI values (in-plane perpendicular directions) and one CCI value. CI and CCI values are typically reported in mm/m. The CCI provides a reasonable approximation of true engineering strain and is acceptable for monitoring in-plane strain. *See* MPR-4273 (INT019)(NP), (INT021)(P).[3]

Measurement of concrete expansion can be approximated by crack width summation. While true engineering strain is represented by the sum of material elongation and crack widths, the crack width term generally dominates the overall expansion. The Seabrook SMP uses CI/CCI to establish baseline strain in ASR-deformed concrete.

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[3] *See also* T. Mohammed, et al., *Alkali-Silica Reaction-Induced Strains over Concrete Surface and Steel Bars in Concrete*, 100 AC/Materials Journal 2 (2003).

Q64. Is the use of CI measurements consistent with ASR best practices?

A64. (MS, SB) Yes. For example, the FHWA Guideline notes that in-plane expansion measurements should be part of any ASR investigation program, stating that it is a "critical parameter in the evaluation of [a structure's] current condition in view of selecting appropriate remedial action," and that it can be used as a way to find "expansion reached to date" in ASR-affected structures. FHWA Guideline at 22 (NER013). This document further indicates that the periodic measurements can be used to "[a]ssess deterioration and expansion level reached to date; [a]ssess current rate of expansion rate, and [a]ssess potential for future expansion." *Id.* at 6. Similarly, the ISE Guideline and work by Somerville note that the CI can provide an indication of expansion to date. Furthermore, work by Mohammed and others found that CI always overestimated the internal strain at the embedded reinforcement.[4]

Q65. Along the same lines, is the use of CCI consistent with ASR best practices?

A65. (MS, SB) Yes, CCI measurements not only quantify the ASR cracking/expansion condition at the surface or near-surface region but also represent the ASR cracking/expansion condition at the depth of the structure at the measured area. The FHWA Guideline notes that surface cracking will develop due to internal expansion even when the surface region is not expanding (such as due to lower moisture levels that do not support ASR). *See* FHWA Guideline (NER013). This was confirmed by testing performed on the concrete cores extracted from a below-grade wall at Seabrook where there is apparent ASR cracking on the concrete surface. More specifically, petrographic examination revealed that the level of ASR-related distress at the near-surface region of the concrete wall is similar, if not larger than that at-depth

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[4] *See* Mohammed et al., *ASR Expansion of Concrete Beams with Various Restrained Conditions—612 Days of Accelerated Marine Exposure*, Proceedings of the 12th International Conference on Alkali-Aggregate Reaction in Concrete, Vol. 2 (2004).

in the wall. *See* Simpson Gumpertz & Heger Inc., Document No. 110594-RPT-02 Rev. 1, "DRI and Visual Assessment of Bravo Electrical Tunnel Core Sections, NextEra Energy Seabrook, LLC, Seabrook, NH" (Feb. 10, 2012) ("DRI & ASR") (NER028).

Q66. How is structural deformation considered in the program?

A66. (MS, SB) At Seabrook, the structures are typically separated by seismic gaps with a designed joint width. Typically, the seismic gaps are filled with seal materials. Visual observations of the seal materials (e.g., whether the seal material is separated from the concrete surface of a specific structure or is torn or cracked in a specific direction), joint width measurements at seismic gaps, or annulus measurements between the Containment Building and the Containment Enclosure Building are performed to investigate whether there is relative movement between the structures. In addition, the visual signs of the relative movement of flex joints or equipment across two or multiple structures are also documented and measured (if applicable).

Structural deformation within the individual structure is visually observed and measured through wall-to-wall distance measurements, plumbness measurements at the wall structural elements, and levelness measurements at the slab structural elements. Structural deformation patterns and locations and magnitudes of critical deformation, within the accuracy of the measurements and structural construction tolerances, are considered in the analysis model.

Q67. How does the program evaluate structural cracking?

A67. (SB, MS) During field observations, in addition to ASR-related cracking, particular attention is paid to the conditions that are related to structural behavior. Field observations identify locations with structural distress, such as types and directions of structural cracking, so that correlations can be made between the analytical results for the in-situ condition and field observations.

Q68. What are pin-to-pin in-plane expansion measurements?

A68. (MS, SB) In addition to the visual CI/CCI measurements, ongoing expansion is monitored using demountable mechanical strain gauges that more precisely measure the distance between gauge pins permanently installed in the concrete. This methodology is described in the FHWA Guideline (NER013) and the ISE Guideline (NER012) in-to-pin in-plane expansion is computed as the change in length-measurement values recorded at different times. Similar to CI measurements, pin-to-pin expansion measurements could be influenced by cracks or deformation from loads *other than ASR*. Figure 6 below shows an example of an in-situ gauge; Figure 7 shows a typical removable strain gauge.

Figure 6

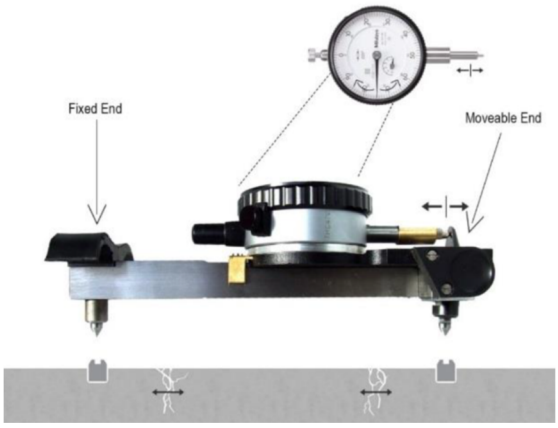

Figure 7

Q69. What are the key differences between CI and pin-to-pin in-plane expansion measurements?

A69. (MS, SB) Over the duration of the monitoring period, pin-to-pin measurements determine changes in ASR expansion more precisely than CI measurements, since they are performed using a calibrated mechanical device capable of measuring changes in length as small as 0.0001 in. However, pin-to-pin in-plane expansion measurements are only able to capture

strains that occur *after* the gauge points are installed. This makes pin-to-pin in-plane expansion measurements ideal for monitoring *changes* in strain. However, CI in-plane expansion can be used to ascertain the existing strains in the concrete *prior* to installation of the pins. Thus, total in-plane expansion can be determined by combining: (1) expansion prior to installation of the pins (i.e., from CI measurements), plus (2) the change in expansion after installation of the pins (i.e., from the pin-to-pin expansion measurements).

Q70. Is all expansion observed via CI measurements and pin-to-pin in-plane expansion measurements caused by ASR?

A70. (MS, SB) Not necessarily. The cracking condition may be complex due to phenomena and loads other than ASR expansion, such as concrete shrinkage or pressurization tests of the CB. Large measurement values therefore may not necessarily imply large ASR expansions.

Q71. Does the SEM account for the possibility that other mechanisms besides ASR may be causing cracking and/or expansion?

A71. (MS, SB) Yes. The fundamental basis for the interpretation of the CI or CCI measurements is the unrealistic, but conservative, assumption that *all* of the cracks measured as part of the CI or CCI are caused entirely by ASR effects. In some situations, this assumption is over-conservative to the point that it limits objective use of the data. Thus, if some or all of the cracks at an ASR monitoring grid are shown to be caused by a mechanism other than internal ASR in the reinforced concrete member, then the CI value or pin-to-pin expansion measurements are adjusted accordingly by excluding the widths of cracks determined not to be caused by ASR. This determination may be through petrography, detailed examination of cracking features, review of operating conditions and history, analytical evaluation for causes other than ASR (e.g., thermal, shrinkage), or other justified means. In essence, the measurement can be adjusted to

account for the non-ASR-related cracking, making it more realistic and representative of actual ASR conditions.

D. Finite Element Modeling

Q72. Earlier, you noted that certain of the analyses employ finite element modeling ("FEM"). What is FEM?

A72. (SB) FEM is a computer analysis method used by engineers to perform complex structural analysis. FEM requires preparing a model (finite element model) to represent the structure. This structural model will include many *elements* and *nodes* that collectively will simulate the structural geometry, stiffness, and mass. There are different types of elements, such as: *beam elements* for representing columns or beams, *shell elements* for representing walls, slabs, and shells, and *three-dimensional solid elements* for representing thick foundations or very thick members. Each structural member, such as a column, is divided into many *beam elements* that are connected together at *nodes* along the height of the column.

Loads such as gravity, wind, or ASR expansion (as measured in the plant) are applied to the FEM to calculate the structural responses. FEM has been a standard tool for engineers to evaluate structural adequacy since the early 1970s. Many standards, such as ASCE 4-16 and the SRP, advocate use of FEM. *See* ASCE/SEI, "Seismic Analysis of Safety-Related Nuclear Structures" (2017) ("ASCE 4-16") (NER036).

Q73. What is the goal of FEM?

A73. (SB) The goal of FEM is to determine the structural forces, stresses, and deformations in the structural elements when required loadings are applied. It provides more precise analytical evaluation of ASR load demands to be added to the original design loads to evaluate total demand on the structure (and for Stage Three Analyses, calculates the total demands due to original design loads as well as self-straining loads including ASR, creep, and

moisture swelling). These total demands are then compared to the original code capacities for use in the structural evaluations.

Q74. Please describe the major geometrical modeling parameters.

A74. (SB) The FEM geometry is based on the structural drawings, and the three-dimensional model of concrete structures uses finite element software element libraries such as: *beam elements* to represent columns and beams, *shell elements* to represent thin walls, slabs, and shells, and *three-dimensional solid elements* for members representing thick slabs and thick walls.

Q75. How does the model treat concrete material properties?

A75. (SB) The model uses the concrete properties specified in the original construction documents, under the proviso that the structure remains within the Expansion Monitoring Limits in the SMP. For seismic Category I structures other than Containment, the concrete member section properties are reduced in accordance with ACI 318-71 procedures to represent cracked section properties for members expected to crack or for structural cracks that are observed during field inspection. The limits at which members are expected to form structural cracks are defined in the ACI 318-71 code (NRC049) and in the SEM.

Q76. What are the ASR load inputs to the FEM?

A76. (SB) The ASR load inputs to these models are: (1) the internal in-plane ASR expansion of reinforced structural members, and (2) the pressure due to ASR expansion of the concrete fill. The internal ASR expansion is determined via the field-measured CI expansion strain; CI is measured in each of the in-plane orthogonal directions. CI represents an equivalent ASR strain. FEM codes do not provide direct inputs for ASR expansion, but thermal expansion can be used as a proxy, and therefore ASR strain is simulated by applying an equivalent thermal load to the concrete only. External ASR pressure may also be exerted by expansion of the

concrete fill (the concrete fill has a similar concrete mix as the structural concrete). The ASR effects of concrete backfill are simulated by applying pressure on adjacent embedded members (Figure 1). Section 4.4.3 of the SEM Document (INT022) provides more detail for defining the pressures due to concrete backfill ASR expansion.

Q77. How are the model results validated?

A77. (SB) In addition to standard checking of the finite element analysis results, we used the results of field observations and measurements to further validate our models (i.e., to confirm that our models actually represent the conditions that are presently observed in situ). The FEM results for in-situ conditions are correlated with the deformations, strains, and distressed areas (if any) observed at Seabrook. More specifically, the FEM is loaded by in-situ loads (loads expected to act in the present condition of the structure) to simulate the structural deformations, strain, and stress distributions in the structure. These loads include gravity loads and hydrostatic loads as defined in UFSAR, ASR field measurements at Seabrook, and ASR pressure from expansion of concrete backfill.

The *simulated* deformations from the FEM are then compared to the *field-measured* deformed shape and maximum deformations. Likewise, the *simulated* stress and strain patterns are compared with *field-observed* distress in the form of structural cracking. These comparisons validate the FEM and confirm its ability to represent the current structural deformed and distressed condition. If needed, further refinement of modeling procedures and/or additional field observations are performed to improve the correlation between analysis results and field observations for locations and types of distress (such as crack type, crack direction, location of cracking region, etc.) and deformation. Section 4.4.4 of the SEM Document (INT022) provides further detail regarding the correlation of the FEM to the field measurements and observations.

E. <u>Structural Evaluations</u>

Q78. Did the LAR request NRC approval of the individual structural evaluations for each seismic Category I structure at Seabrook?

A78. (SB, GB) No. The LAR did *not* seek NRC approval for the individual structural evaluations. Rather, the LAR sought approval of the generic *methodology* for the structural evaluations (i.e., the SEM) and the corresponding changes to the UFSAR. NextEra submitted one of the structural evaluations merely as an example of what an evaluation per the SEM would look like. *See* Rev. 0 CEB Evaluation (INT015).

Q79. Per the SEM, how is the Containment Building evaluated?

A79. (GB) The Containment Building is analyzed and evaluated per Stage One analysis (*see* A50) of the SEM for original design loads and ASR load effects using the structural acceptance criteria for evaluation of the Containment Building in accordance with UFSAR Section 3.8. The basis for the acceptance criteria is ASME 1975 (NRC050).

Q80. How are seismic Category I structures other than Containment evaluated?

A80. (SB) Structural acceptance criteria for evaluation of other seismic Category I concrete structures (including the foundations) are in accordance with UFSAR Section 3.8.4.5. The basis for the acceptance criteria is UFSAR Section 3.8.4.5 and the ACI 318-71 Code (NRC049) as supplemented by mark-up of Seabrook's UFSAR pages. *See* UFSAR Markup (INT010) (NP); (INT011) (P); (NRC089). Structures are evaluated per the SEM to confirm that the structural demands for factored load combinations (for both normal load conditions and unusual load conditions), including ASR loads, are less than the code-defined strength (including the code-defined strength reduction factors).

The acceptance criteria for seismic gaps between adjacent structures are reduced to the field-measured values (when structural evaluation is performed), and could be less than the

original defined gap width in the structural drawings due to current structural deformations caused by ASR expansion. The maximum displacements of adjacent structures, due to a design seismic event factored load combination including factored ASR loads, must remain below the measured (revised) gap between the structures.

Q81. How does the SEM account for future growth of ASR?

A81. (SB) All loads except the load associated with concrete ASR expansion are defined and do not increase in the future. However, the ASR loads will likely increase until the concrete alkali-silica reactivity is completed (which may be beyond the licensed life of the plant). In order to account for future ASR growth, a "threshold factor" is used to linearly increase the current ASR loads. This approach is conservative because future ASR growth is expected to slow down, since the amount of reactive material decreases over time. As noted above at A45, the threshold factor is the current design margin expressed as the amount which ASR loads can increase linearly beyond measured values such that the structure and structural components will still meet the allowable limits (capacities or strengths) defined by the code.

Q82. Does the SEM seek to evaluate the structures for the full potential ASR expansion?

A82. (SB) No. The seismic Category I structures are not evaluated for the purpose of calculating the time at which ASR expansions are completed. These structures are evaluated conservatively until the ASR expansion for each building reaches an applicable threshold factor for ASR expansion. All structures are being monitored against these threshold factors, which are essentially acceptance criteria. The threshold factors calculated for different seismic Category I structures range from 20% to more than 100% above the current measures of ASR expansion. For example, the Rev. 0 CEB Evaluation (INT015) shows a threshold of 1.2 (i.e., 20% beyond

current measured ASR values). A larger threshold factor means a larger reserve margin between the current measurement of ASR expansion and the code limit.

Q83. How are the structural monitoring parameters defined?

A83. (SB, MS) The threshold factor informs the definition of required monitoring parameters and their limits, as well as the selection of observations for each building. Strains are always monitored, and structure deformations are monitored when applicable. Strains are used as input to the model and can be monitored by CI, CCI, or pin-to-pin expansion measurements. Deformations are calculated by finite element analysis and can be monitored by field measurements of the width of seismic isolation gaps between the structures, distances between adjacent structures, and localized structure deformations. Monitoring parameters could also include field observation of structural cracking. The LAR defines the frequencies of thirty-six, eighteen, and six months, depending on analysis conservatism and margin remaining for each building at current conditions. However, the monitoring period could be more frequent as conditions dictate.

As described in the Rev. 0 CEB Evaluation (INT015), the CEB is divided into regions, and the monitoring consists of measurements of regional strain as represented by CI, CCI, and deformation measurements of isolation gaps and annulus distance between the CEB and Containment Building. Measurements are taken at multiple locations within a region, and the average value is used and compared to the appropriate threshold limit. These parameters, limits, and observations were defined based on structural evaluation calculations to make sure the building deformations and distressed conditions remain within the evaluation conditions and meet the code requirements.

V. DISCUSSION OF SPECIFIC CLAIMS IN DR. SAOUMA'S TESTIMONY

A. Selection of Methodology

Q84. Dr. Saouma suggests NextEra, in its LAR, should have used a "state of the art" approach, such as the "constitutive modeling" approach he developed. *See, e.g.*, Saouma Testimony at 31 (INT001-R); *see also generally* V. Saouma, Final (Public) Summary Report (INT005). Are you familiar with constitutive modeling?

A84. (SB, MS) Yes. Constitutive modeling, as used here (also referred to as chemo-mechanical modeling, as discussed in the MPR Testimony at A195, A198 (NER001)), is a technique that models how ASR formation occurs within the concrete and how the resulting effects change the loads and capacities within the modeled system. Specifically, it represents the relations between the ASR mechanism, stress (force per unit area) components, and strain (expansion per unit length) components. There are many constitutive modeling techniques that attempt to address ASR, but there is no industry consensus on an appropriate methodology to use. Esposito and Hendriks summarized forty different formulations developed by different authors for simulating ASR-affected concrete structures; the authors also categorize these formulations based on the number of input parameters and show that many parameters are required for most of these formulations. *See* R. Esposito & M.A.N. Hendriks, *Literature Review of Modelling Approaches for ASR in Concrete: A New Perspective*, EUROPEAN JOURNAL OF ENVTL. AND CIVIL ENG'G (2017) ("Esposito & Hendricks") (NER037). Other authors, have binned the methodologies into the following types:

- Micro-models: These aim to capture microscopic scale modeling of the gel behavior, the cementitious materials, and aggregates.[5]

[5] *See* Z.P. Bažant and A. Steffens, *Mathematical Model for Kinetics of Alkali-Silica Reaction in Concrete*, 30 Cement and Concrete Research 3 (2000).

- Meso-models: These consider the local interaction of ASR gel and the concrete to simulate the resulting deformations and cracking of the concrete microstructure.[6]

- Macro-models: These models intend to incorporate ASR behavior in the constitutive models that can be used for structural-scale modeling. However, there are many different formulations of macro-models.[7] Cope proposed stress formulation and independent formulation in each direction.[8] Charlwood uses a logarithmic expansion rate and stress, while Cope uses a linear expansion rate and stress.[9] Both use a limiting compression stress that can be developed by ASR expansion, but there is no consensus on the magnitude of this parameter, with recommended values ranging from 5 MPa to 20 MPa. This is a very wide range that could significantly impact the structural simulation. There are also many researchers focusing on volumetric ASR expansion, including coupled mechanical and ASR constitutive models.[10]

Q85. What is your general assessment of these constitutive modeling approaches?

A85. (SB, MS) There are significant drawbacks to each category. For example:

- Micro-models: This type involves extremely detailed modeling, requiring very fine mesh of less than 1 inch. It is definitely not suited for structural modeling.

- Meso-models: These models, which still require very refined mesh, may be suited for simulating a small localized area but are not suited for simulating overall structural behavior and interaction of different structural components.

- Macro-models: These models are intended to simulate the structural behavior from the time the alkali and silica start to react to the time the ASR slows or ends. These

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6 *See* C.F. Dunant and K.L. Scrivener, *Micro-Mechanical Modelling of Alkali-Silica-Reaction-Induced Degradation Using The AMIE Framework*, 40 Cement and Concrete Research 4, 517-25 (2010).

7 *See* R.G. Charlwood, *A Review of Alkali Aggregate Reaction in Hydro Plants and Dams*, 1 Int'l Journal on Hydropower & Dams 3, 73-80 (May 1994) ("Charlwood").

8 And R.J. Cope, et al., *Prediction of Stress Distributions in Reinforced Concrete Members Affected by Alkali Aggregate Reaction*, Project Report 44, Transportation Research Laboratory, Crowthorne, U.K. (1994)

9 *See* Charlwood.

10 *See, e.g.*, F. Ulm et al., *Thermo-Chemo-Mechanics of ASR Expansion in Concrete Structures*, 126 ASCE Journal of Eng'g Mechanics 3, 233-42 (2000); E. Fairbarin et al., *Modeling the Structural Behavior of a Dam Affected by Alkali-Silica Reaction*, 22 Communications in Numerical Methods of Eng'g 1, 1-12 (2006); E. Grimal et al., *Creep, Shrinkage, and Anisotropic Damage in Alkali-Aggregate Reaction Swelling Mechanism Part I: A Constitutive Model*, 105 ACI Materials Journal 3, 227-35 (2008); C. Comi et al., *Anisotropic Damage Model for Concrete Affected by Alkali-Aggregate Reaction*, 20 Int'l Journal of Damage Mechanics 4, 598-617 (2011); and V. Saouma et al., *Constitutive Model for Alkali-Aggregate Reactions*, 103 ACI Materials Journal 3, 194-202 (2006); D.M. Wald, ASR Expansion Behavior in Reinforced Concrete—Experimentation and Numerical Modeling for Practical Application (Aug. 2017) (unpublished doctoral thesis, University of Texas at Austin) https://repositories.lib.utexas.edu/handle/2152/61820; J.W. Pan, et al., *Modeling of alkali-silica reaction in concrete: a review*, 6 Frontiers of Structural and Civil Eng'g 1 (2012); and A. Jurcut, A., Modelling of Alkali-Aggregate Reaction Effects in Reinforced Concrete Structures (2015) (unpublished masters thesis, University of Toronto) http://www.vectoranalysisgroup.com/theses/Jurcut-MASc(2015).pdf.

> models are non-linear and analytically are very complex; therefore, significant forward-looking and backward-looking validations are required before being suitable to use for practical evaluation such as at Seabrook. These models require significant data collection from the time that construction was completed, such as humidity, temperature, external climatic conditions, localized moisture migration, and structural deformation, to attempt to simulate the observed current condition at Seabrook.

Furthermore, these techniques have been reviewed by others, such as the authors of the CSA Guideline who concluded:

> Micro-models…focus on the physical-chemical principles to simulate the AAR reaction kinetics and concrete expansion…Generally, the effects of restraints, as well as moisture transport, are affected by external climatic conditions that are not taken into consideration. This is a major disadvantage of these models, which makes them less attractive for civil engineering applications.

CSA Guideline at 36 (NRC076). The CSA Guideline also states that "[d]ue to the complexity of the AAR expansion process, any AAR simulation model, as sophisticated as it might be, must be calibrated against the reversible and irreversible displacement recorded from the monitoring system and, if possible, the strains in reinforcing steel and concrete." *Id.* at 37.

Similarly, the FHWA Guideline notes that "[d]ue to the complexity of the ASR expansion process, any simulation model, as sophisticated as it might be, must be calibrated based on the monitoring data and pertinent information obtained from in-situ and laboratory investigations." FHWA Guideline at 28 (NER013).

A report describing analyses performed regarding the Gentilly-2 nuclear power plant in Quebec, notes that "[materials tests] cannot, however, be simulated as initial boundary-value problems …that, even though conceptually attractive cannot be employed in the context of analysis of large-scale structures." V. Gocevski, *Pathologies/Degradation Mechanisms Experienced by Hydro-Quebec During the Evaluation of Gentilly-2 NPP*, Report Submitted to ASCET, (June 2015) (NER038).

Lastly, Esposito and Hendriks note that:

> the use of phenomenological approaches is risky as long as the interactive multiscale nature of ASR is not yet understood," […] "More effort is now required in order to upscale these models for structural analyses," […] "computational approaches, which discretize the concrete constituents with FEM, perform detail simulation of the damage at aggregate level which can be compared with microscopic observations…but their abilities to upscale to structures appear limited."

Esposito & Hendricks at 18.

Q86. Was constitutive modeling considered in lieu of maintaining the approach from the original design and licensing basis?

A86. (SB, MS) Yes. However, we concluded that it was not an appropriate solution for Seabrook for several reasons. Most notably, the fundamental approach of constitutive modeling requires starting at time zero, modeling all reactions and effects that have occurred to date, calibrating the model against current conditions, and then using the model to project forward to an unknown point in time. Instead, the analysis approach ultimately selected by NextEra was to establish the bounds within which the code-based UFSAR equations and procedures for calculating concrete strength remain valid (i.e., the LSTP); monitor Seabrook's concrete to ensure it remains within those bounds (i.e., the SMP); and analyze the structures accordingly (i.e., the SEM).

Furthermore, constitutive modeling is inappropriate for application to an operating nuclear power plant because:

(1) even "state of the art" constitutive modeling is too immature for practical immediate application;

(2) there is no industry consensus for how it should be applied;

(3) variations in opinion among experts can lead to widely varying results;

(4) there is no means at present to relate the results of constitutive modeling to accepted performance criteria; and

(5) the most direct, meaningful, and reliable solution is to base structural evaluations on the requirements of the original design basis.

In summary, the micro- and meso-models are not suited for structural-scale modeling, and there are too many macro-modeling techniques in research stages without consensus on any one method. The macro-constitutive modeling is nonlinear, is analytically very complex, requires significant data not available from the time of construction, needs significant validation before being suited to use for nuclear facilities such as Seabrook, and are typically used to extrapolate to the ultimate ASR expansion – a degree well beyond that anticipated or permitted by the Expansion Monitoring Limits in the SMP.

These complex constitutive models would introduce significant uncertainties into the licensing basis and would have necessitated large departures from the methodology of the original design basis documents—in essence, *entirely abandoning the codes of record*. Moreover, to date, no approaches using constitutive models have gained industry-wide acceptance or have been endorsed by national codes or design specifications for civil/structural applications. And finally, the use of *any* modeling technique, including constitutive modeling, to evaluate an existing structure requires validation against the current condition of the structure. Thus, from an efficiency perspective, modeling presents no benefit over an approach (such as the SEM) that employs actual plant measurements. Accordingly, NextEra did not pursue the constitutive modeling approach.

Q87. Dr. Saouma has testified that the SEM should have employed a PRA. Are you familiar with PRA methodology?

A87. (GB, SB) Yes. SGH is a leader in the use of PRA methods for structure, system, or component ("SSC") evaluations in the nuclear power industry. SGH implements PRAs frequently in circumstances where they are appropriate and present the best method for a particular evaluation. SGH engineers have authored numerous technical papers on the use of

PRA methods to assess the risks to nuclear plants from natural hazards such as earthquakes and wind.

Q88. Please describe PRA.

A88. (GB, SB) PRA is a systematic and comprehensive methodology to evaluate risks associated with a complex engineered technology. PRAs quantify risk based on:

- the magnitude of the possible adverse consequences, and
- the probability of occurrence of these consequences

In the nuclear power industry, PRA is a tool used to determine the risk of failure of a plant SSC and ultimately to assess the risks to the safe operation of a nuclear power plant. Risk of failure is computed considering the probabilistic variability of all of the factors that determine the hazards (i.e., loads or demands) and vulnerabilities (i.e., failure modes and limits on capacity).

Q89. Did the SEM use PRA?

A89. (SB, GB) The SEM considered the *variability* in ASR loading in determining the loads and load factors for use in the analysis, but the methodology is not a PRA *per se*. Rather, as described earlier, the SEM is founded on the original design basis outlined in the UFSAR, which is a standards- and code-based approach as recommended and approved by the NRC in the SRP. More specifically, the NRC endorsement of this approach is in SRP § 3.8.3 for the concrete containment, SRP § 3.8.4 for Seabrook's other seismic Category I structures, and SRP § 3.8.5 for foundations.

Q90. Why is the SEM based on the standards- and code-based approach rather than a PRA?

A90. (SB, GB) Nuclear plants are licensed to operate based on their licensing basis. This licensing basis includes a set of codes and standards which serve to ensure that the plant

will operate safely over the course of its life. When questions arise associated with the safety of plant structures, the primary means to assess the safety impact is to assess whether any change or question violates the plant licensing basis. Thus, assessing an impact to the plant licensing basis is the most direct and reliable means of assuring that the level of safety and reliability of the plant implicit in the original design criteria has not—and will not—be compromised by the presence of ASR. This approach conforms to the UFSAR, which is the legal basis for the license for the plant.

PRAs are more appropriately used in more extreme cases where new hazards compromise the original design basis of the plant. This has not been necessary at Seabrook, in part, because the ASR at Seabrook is slow-growing and, after more than thirty years, has reached only low to moderate levels. Consequently, evaluating the safety and reliability of the SSCs against the original design criteria by considering the *incremental* effects of ASR in loads is far more desirable (and from a licensing standpoint, more direct) than abandoning the original design criteria altogether and embarking on a new and different approach for which there are no established acceptability criteria.

Q91. In Section C.3.4.1.1, Dr. Saouma argues that a probabilistic based analysis should be performed for structural evaluations at Seabrook. *See* Saouma Testimony at 29-30 (INT001-R). Do you agree?

A91. (SB, GB). No. Seabrook seismic Category I structures are evaluated on a standards-and-code basis to be consistent with the original design basis of the plant as described in the UFSAR. The code-based SEM is more direct and reliable than PRA since it ensures that the originally intended reliability of the Seabrook structures will not be compromised and it is based on codes, standards, and methods that have been time tested for decades. As explained in A89 and A90, we determined that the code-based approach was superior to the PRA approach for various reasons. Nothing in Dr. Saouma's testimony convinces us otherwise.

Q92. In his testimony Dr. Saouma criticizes the SEM's use of linear, elastic analysis as simplistic and inappropriate. For example he states "Analysis for design of new structures starts by amplifying the load by say 40 or 50%, and the response up to failure is assumed to be linear (this is indeed code-driven). In analyzing the safety of existing safety structures, one has to determine the exact nonlinear response beyond the elastic limit and – most importantly – determine the corresponding deformation that would be under-estimated in the former case." Saouma Testimony at 7 (INT0010-R). How do you respond?

A92. (SB, GB) In selecting analytical techniques, one must use the technique most appropriate to the methodology and goals of the analysis, and more specifically, when evaluating the Seabrook structures relative to the original design basis, one must use analytical techniques that are consistent with the prescribed codes. As mentioned earlier, these are ASME 1975 (NRC050) (for the Containment Building) and ACI 318-71 (NRC049) (for Seabrook's other seismic Category I structures). Both employ linear, elastic behavior.

Q93. What do you mean by linear, elastic behavior?

A93. (GB, SB) Figure 8 is a graph depicting the general load-deformation behavior of a reinforced concrete structure (this could represent the response under load of a component of the structure or of the structure as a whole). As load increases, the structure deforms (i.e., displaces from its unloaded state) along the line shown in the graph. Initially, the deformation increases proportionately to the load. In other words, the load-deformation relationship is a straight line, or linear.

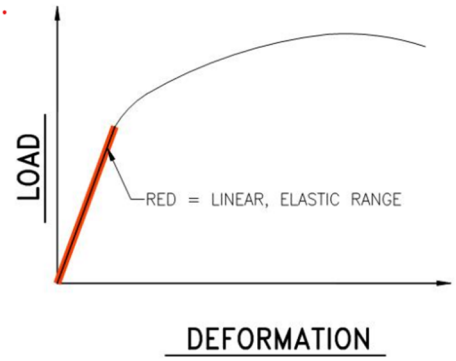


Figure 8 - General Load-Deformation Behavior of a Reinforced Concrete Structure

If, while still in the range of linear behavior as shown by the red line, the load is removed, the structure recovers along the same straight line to a point of zero deformation. The structure's recovery to zero deformation is called elastic behavior. (Note that in this description, concrete is assumed to have zero tensile strength and so is cracked when experiencing tensile stress).

If the structure is loaded beyond the limits of linear elastic behavior, its deformation increases faster than the load increases. This is called non-linear behavior, and is represented by the curved part of the load-deformation curve. The point at which the behavior becomes non-linear in reinforced concrete corresponds to the onset of yielding of the reinforcement steel accompanied by increased cracking of the concrete. If the structure continues to be loaded into the non-linear range, the load resistance eventually reaches a maximum peak, and then the load resistance declines.

Q94. Please describe the relevance of linear, elastic analysis with respect to the requirements of ASME 1975 for the Containment Building.

A94. (GB, SB) ASME 1975 requires demonstration of structural adequacy of the Containment Building under so-called "factored load conditions" and "service load conditions." Factored load conditions:

> are load combinations, including multipliers or conditions resulting from a postulated single failure of the reactor coolant system or those environmental conditions postulated as upper bound limits for the plant site. Also included in this category are load combinations, with other multipliers, of a single failure of the reactor coolant system plus severe or extreme environmental conditions.

ASME 1975 ¶ CC3132 (NRC050). ASME 1975 imposes limits on stress under factored load conditions to keep the containment below the level of general yielding of the reinforcement. This is within the linear-elastic range of behavior as shown in Figure 9.

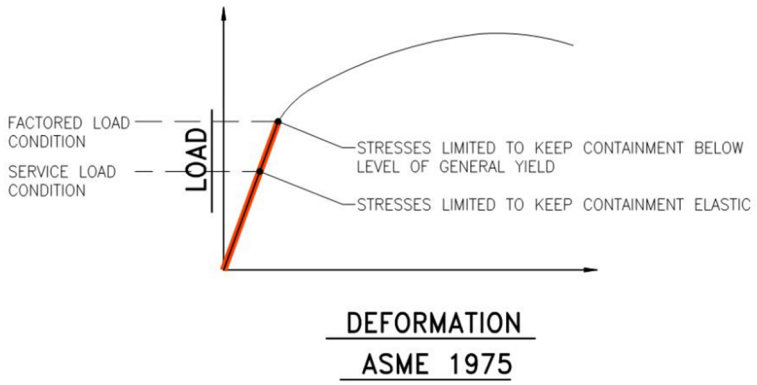


Figure 9 - Load and Performance Criteria of ASME 1975 (NRC050)

"Service Load Conditions" are "any conditions encountered during construction and in the normal operation of a nuclear power plant." ASME 1975 ¶ CC3131 (NRC050). In order to maintain the integrity of the containment as a pressure-bearing vessel, stresses under service load conditions are limited to keep the containment structure well into the elastic range.

For analysis under both Service and Factored Load Conditions, ASME 1975 § CC-3300 contains requirements for analysis procedures. Paragraph CC-3310 states "Methods of analysis which are based on accepted principles of engineering mechanics and which are appropriate to the geometry of the building shall be used." ASME 1975 ¶ CC-3310 (NRC050). The linear-elastic method was the generally-accepted method of analysis for design at the time of Seabrook's design and remains so today. Paragraph CC-3320 states "Elastic behavior shall be the accepted basis for predicting internal forces, displacement and the stability of thin shells."

Id. at ¶ CC-3320. Paragraph CC-3330 states that for Base Mat, Frames, Box Type Structures, and Assemblies of slabs, "Analyses based on elastic behavior, or other methods generally accepted in conventional practice, shall be used." *Id.* at CC-3330.

Q95. Please describe the relevance of linear, elastic analysis with respect to the requirements of ACI 318-71 for Seabrook structures other than the Containment Building.

A95. (GB, SB) The structural performance criteria of ACI 318-71 are similar to ASME 1975 in that strength and serviceability are checked at two levels: Factored Load Conditions and Service Load Conditions, respectively, as shown in Figure 10.

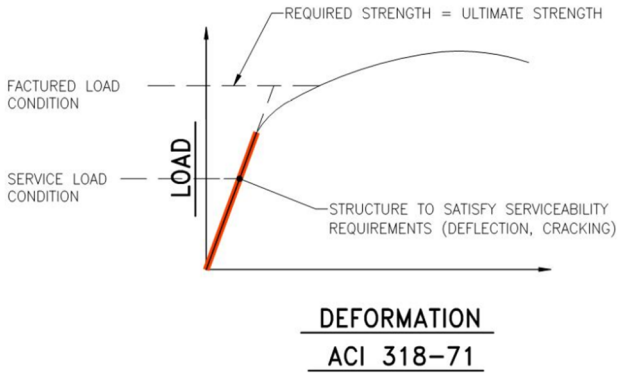


Figure 10 - Load and Performance Criteria of ACI 318-71 (NRC049)

Since the other seismic Category I structures do not have the same pressure-bearing function as the Containment Building, the requirements of ACI 318-71 are focused principally on adequate strength. The capacity of the member, as determined by its "ultimate strength" through code equations, must be equal to or greater than the demand under Factored Load conditions. Performance at the ultimate strength level generally involves cracking of concrete, with reinforcement resisting the tensile force. There are also requirements controlling "serviceability" performance at Service Load Conditions. ("Serviceability" requirements are generally acceptance criteria for performance other than strength. This is described more fully in A119).

Regardless of the differences in performance criteria between ASME 1975 and ACI 318-71, the ACI Code is based on linear-elastic analysis with some force and moment redistribution allowed. Consistent with the Code, the SEM employed cracked section properties where appropriate. Paragraph 8.4.1 of ACI 318-71 states "All members of frames or continuous construction shall be designed for the maximum effects of the design loads as determined by the theory of elastic frames." ACI 318-71 ¶ 8.4.1 (NRC049). Paragraph. 19.3.1 states "Elastic behavior shall be the accepted basis for determining internal forces, displacements, and stability of thin shells." *Id.* at ¶ 19.3.1. Similar to ASME 1975, ACI 318-71 is very specific in its stipulation of elastic analysis.

Q96. How do these requirements in ACI 318 and ASME 1975 pertain to Dr. Saouma's criticism regarding the SEM's use of linear, elastic analysis?

A96. (GB, SB) Dr. Saouma's criticism disregards the fundamental analytical approach embedded in Seabrook's UFSAR (flowing from ACI 318 and ASME 1975) that Seabrook structures should be analyzed through elastic behavior. In fact, as noted in the NRC's Final SE:

> As required by the structural design in the Seabrook UFSAR Sections 3.8.4.3 and 3.8.4.5 (corresponding UFSAR subsections for containment internal structures are 3.8.3.3 and 3.8.3.5), stresses and strains in the structures *shall be maintained within elastic limits* under normal operating (service) load conditions.

Final SE at 48 (INT024)(NP), (INT025)(P)).

In other words, to comply with Seabrook's licensing basis, loads, analytical techniques, and determination of performance against acceptability criteria all go hand-in-hand and must be in accordance with the codes and standards detailed in the UFSAR that form the licensing basis for Seabrook. Thus, the non-linear analysis Dr. Saouma demands is not applicable or relevant to structural behavior at Seabrook.

Q97. Dr. Saouma also claims that "A simplified linear elastic analysis (used in design of new structures) will under-estimate the displacements and cannot capture either the failure load or the deformation. Safety assessment can only be performed through a non-linear analysis." Saouma Testimony at 7 (INT001-R). What is your response?

A97. (SB, GB) The purpose of the structural analysis is *not* to determine the failure loads of structural elements. This is unnecessary because laboratory research (i.e., the LSTP) has demonstrated that code equations and procedures in combination with the use of the originally-specified concrete and reinforcing steel material properties are reliable and conservative predictors of the strength of ASR-affected structural elements within the limits of ASR in the laboratory specimens. Neither is the purpose of the SEM to predict the ultimate deformations of the structure. As stated earlier, the primary purpose of the structural analysis is to determine the demands (loads) on structural components. With this goal, a determination of internal or external ASR loads is more conservative (i.e., results in higher load demands) with linear, elastic analysis than for nonlinear analysis because the structure is stiffer in the linear-elastic range than in the nonlinear range (a structure is "stiffer" if it requires more load to produce a given degree of deformation or movement than a structure that is "softer"). As illustrated in Figure 11 below, for a given amount of deformation, the load determined by a fully elastic analysis will be greater than the load determined from an analysis where the structure "softens" in the nonlinear range.

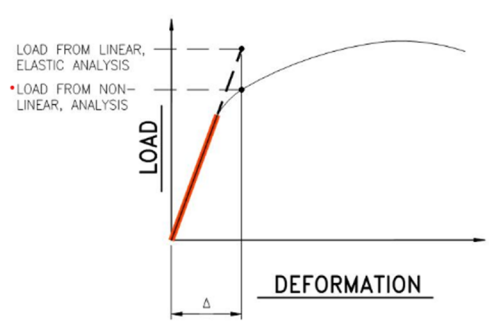


Figure 11 - Loads Determined from Linear-Elastic vs. Non-linear Analysis

B. <u>Dr. Saouma's Criticisms of the Rev. 0 CEB Evaluation</u>

Q98. In his testimony, Dr. Saouma provides a list of 18 items that he asserts as "concerns" regarding the Rev. 0 CEB Evaluation (INT015). *See* Saouma Testimony at 24-29 (INT001-R). Do any of these "concerns" identify a deficiency in the LAR?

A98. (SB, GB) No. As noted in A78 above, the LAR did *not* seek NRC approval of the Rev. 0 CEB Evaluation (INT015). The LAR only sought approval of the SEM (INT022) and the corresponding changes to the UFSAR. Thus, Dr. Saouma's criticisms of the Rev. 0 CEB Evaluation do not directly challenge the sufficiency of the LAR.

Furthermore, the Rev. 0 CEB Evaluation critiqued by Dr. Saouma employed a *draft* version of the evaluation methodology. Subsequent discussions between NextEra and the NRC Staff led to various refinements in the methodology, which are reflected in the final version of the SEM approved by the NRC. For example, as noted in the Final SE, the Rev. 0 CEB Evaluation used the "100-40-40" approach for combining the effects of seismic loading in three directions, but NextEra subsequently revised the CEB Evaluation to use the "Square Root of Sum Squares" method. *See* Final SE at 41 (INT024)(NP), INT025(P). Likewise, NextEra subsequently revised the CEB Evaluation to consider cracked section properties instead of the moment redistribution method used in Revision 0. *See id*. at 39-40. Thus, generally speaking, Dr. Saouma's criticisms are outdated and do not challenge the (final) SEM.

And as discussed in our responses to the specific questions below, none of his individual criticisms of the Rev. 0 CEB Evaluation have merit.

Q99. In his testimony, Dr. Saouma suggests that the use of a linear elastic analysis in the Rev. 0 CEB Evaluation is inappropriate. *See* Saouma Testimony at 7, 24 (item 1), 28 (item 16), and 28-29 (item 18) (INT001-R). How do you respond?

A99. (SB, GB) As noted above in A91 through A96, to properly evaluate the Seabrook structures against the original design basis, one must use methods of analysis that are consistent

with the prescribed acceptance criteria. Linear, elastic analysis is the accepted basis for evaluation according to ASME 1975 (NRC050) and ACI 318-71 (NRC049). Moreover, it conservatively predicts the load effects due to ASR at Seabrook. Thus, linear, elastic analysis is entirely appropriate for application to Seabrook.

Q100. The Rev. 0 CEB Evaluation (INT015) states that "ASR expansion impacts the total demand on reinforced concrete elements, but does not reduce the resistance (capacity) of reinforced concrete elements so long as the strain does not exceed the limits defined in [MPR-4273]." Dr. Saouma argues this is incorrect because: (1) the concrete material is degraded by ASR, (2) SGH confuses structural testing with material testing, and (3) linear elastic analysis grossly underestimates strains. *See* Saouma Testimony at 25-26 (item 8) (INT001-R). How do you respond?

A100. (SB, GB) Regarding Dr. Saouma's assertions (1) and (2), and as more fully described in A40 and A75, SGH completely understands the difference between material behavior (and degradation potential) and structural element behavior. While ASR may degrade both the strength and stiffness of the *unconfined* concrete material, the research has demonstrated that, within certain ASR strain limits (*see, e.g.*, MPR-4273 (INT019)(NP); (INT021)(P)), neither the strength nor the stiffness of structural elements is degraded below that predicted by code equations and principles of structural mechanics if original concrete properties are used. *See also* MPR Testimony at A91 (NER001).

In fact, this is a well-established concept in ASR structural engineering. For example, the ISE Guideline notes that:

> [i]t is emphasised that the residual strength and stiffnesses in actual structures will be modified from the figures in Table 4 [which show reductions in properties due to ASR]. This is because the concrete in actual structures is generally restrained by adjacent material and is in a biaxial or triaxial stress state. These effects will tend to reduce the damage to the concrete and increase its residual mechanical properties.

ISE Guideline at 14 (§ 4.4) (NER012).

In summary, the accurate structural member capacities and stiffnesses are those determined by code equations and principles of structural mechanics using the originally-specified concrete strength. Demands are determined using linear, elastic analyses per the original design basis and referenced codes and standards. The analysis properly compares the structural capacities (e.g., strengths) to the code- and standard-required demands (load effects) to assure structural safety within the objectives of the original design basis.

Regarding Dr. Saouma's assertion (3), that linear elastic analysis grossly underestimates strains, we again note that the purpose of the SEM is not to *predict* future ASR strains. Rather, the Expansion Monitoring Limits in the SMP guard against unacceptable future ASR strain. This is an important feature of the LAR that should not be overlooked.

Q101. Dr. Saouma also claims that "[t]he assumption that ASR can be considered a load is fundamentally wrong," Saouma Testimony at 17 (INT001-R), and that "NextEra is confusing Capacity with Demand," *id.* at 24 (item 2). How do you respond?

A101. (SB, GB) A "load," as that term is used in building codes, is described in A51. Internal loads develop in reinforced concrete members affected by ASR. Also, force is developed in concrete members affected by ASR with external restraints. These loads must be accounted for in the structural evaluation. *See* A52 through A54. Similarly, as described in A49 and A76, the expansion of the concrete backfill placed between the excavated bedrock and structures at Seabrook will create loads as it expands against the existing structures. These loads must also be accounted for in the structural evaluation. If ASR only changed material properties and not load, as asserted by Dr. Saouma, then there would be no displacements or deformations associated with ASR. However, actual field observations at Seabrook entirely disprove Dr. Saouma's theory because displacements and deformations have, in fact, been observed. In other

words, our approach to loading and modeling is *validated by corroboration* of the analytical results with *actual field observations*.

Q102. Next, Dr. Saouma points to a statement on page 16 of the Rev. 0 CEB Evaluation (INT015) regarding the 1.2 "threshold factor" from the assessment and asks "on what basis is it assumed that future expansion will increase by 20%?" Saouma Testimony at 24-25 (item 3) (INT001-R). What is your response?

A102. (SB) Dr. Saouma incorrectly characterizes the "threshold factor" as an *assumption*. As explained previously, the threshold factor is building-specific and is a calculated value, not an assumed value. It represents the amount that presently identified ASR load can be increased such that the structure and structural components will still meet the allowable limits of the code expressions (*see* A45 and A81). As discussed in A82, the threshold factor is *not* intended to calculate the final ASR growth. It is used as a monitoring benchmark for future expansion.

Q103. Furthermore, Dr. Saouma challenges NextEra's assumption, in the Rev. 0 CEB Evaluation (INT015), that the elastic modulus of concrete is not reduced due to ASR damage, *see* Saouma Testimony at 25-26 (items 4, 6, 8) (INT001-R), and more broadly claims that NextEra confuses material properties with structural strength, *see id.* at 17. In fact, he asserts "[t]here is *no doubt* that the elastic modulus E is affected by ASR." *Id.* at 25 (item 4) (emphasis added). Is his criticism valid?

A103. (SB, GB) No. For the reasons described below, this is not a valid criticism. Laboratory research (i.e., the LSTP) has shown that, within certain strain limits, the stiffness and strength of reinforced concrete members are not reduced by ASR (as compared to the stiffness calculated from standard principles of structural mechanics and strength determined by analyses involving accepted codes and standards) when the original design concrete strength and material properties are used (A75). The material properties of *unconfined* concrete affected by ASR may be reduced. But because of the interaction between concrete and reinforcing in a reinforced concrete member, the strength and stiffness of the overall members are not reduced within

certain ASR strain limits. At Seabrook, strain limits are incorporated as acceptance criteria (i.e., the Expansion Monitoring Limits) for the expansion monitoring aspect of the LAR in the SMP.

Q104. Does Dr. Saouma point to any evidence to support his claim to the contrary (i.e., that elastic modulus in reinforced concrete is *always* degraded by ASR), such as independent testing he performed or other academic literature directly on point?

A104. (SB, GB) No. Dr. Saouma does not provide any evidentiary support for his claim.

Q105. Dr. Saouma quotes an excerpt from the Rev. 0 CEB Evaluation (INT015) noting that "[t]he magnitude of ASR expansion . . . is based on field measurements," and he comments that "this is where monitoring and structural assessment intersect," and points to his challenge to the representativeness of the LSTP. Saouma Testimony at 25 (item 5) (INT001-R). Do you understand his criticism?

A105. (SB, GB) No. His precise criticism is unclear to us. The quoted statement from the Rev. 0 CEB Evaluation (INT015) pertains to measurements taken at Seabrook; whereas, his comment about the "representativeness of tests" pertains to the LSTP. As best we can determine, he is conflating the LSTP with "field measurements" taken on Seabrook concrete. The representativeness of the LSTP is addressed in Section VII.B of the MPR Testimony (NER001).

Q106. Dr. Saouma quotes an excerpt from the Rev. 0 CEB Evaluation (INT015) noting that "the same aggregate source was used for the concrete fill as for the CEB concrete" and claims this statement is "incorrect" pursuant to a purportedly contrary statement in MPR-4273, Rev. 1 (INT021). *See* Saouma Testimony at 25 (item 7) (INT001-R). Is he correct?

A106. (SB, GB) No. The language Dr. Saouma quotes from MPR-4273, Rev. 1 (INT021), describes the aggregate source for the LSTP; whereas the language he quotes from the Rev. 0 CEB Evaluation (INT015) describes the aggregate source for the CEB and its surrounding concrete backfill. As shown in Figure 1 above, Seabrook was built into excavated bedrock, and its construction used concrete as backfill between the final structure and the excavation face.

The concrete backfill used the same mix as that used for the reinforced concrete members of the Seabrook structures that are affected by ASR expansion.

The language quoted by Dr. Saouma from the Rev. 0 CEB Evaluation is merely noting that the backfill concrete is of the same type and composition as the CEB's structural concrete. As best we can tell, Dr. Saouma simply confuses the term "concrete fill" as somehow referring to the LSTP—which is incorrect.

Q107. Dr. Saouma criticizes the use of "shell elements" in the CEB finite element analysis. *See* Saouma Testimony at 26 (item 9) (INT001-R). More specifically, he asserts that shell elements "cannot capture the through thickness expansion." *Id.* Is this a valid criticism?

A107. (SB, MS). No. The concern is unfounded. As explained in further detail below, the use of shell elements is appropriate for the CEB because the in-plane forces and flexural moments are the dominant loadings, and shell elements are appropriate for the analysis of these loadings, because:

(1) The primary response of a shell structure is to transfer forces via in-plane actions and out-of-plane flexure. Thus, the use of shell elements to model the CEB is appropriate to capture the structural deformation of the two-dimensional in-plane behavior that is relevant to the CEB thin-walled structure. *See* ASCE 4-16 ¶¶ 3.1.3.1 & 3.1.3.2. (NER036).

(2) The CEB is a relatively thin-walled structure (3 ft-0 in. at the base (EL (-)30 ft-0 in.), transitioning to 2 ft-3 in. at EL (-)11 ft-0 in. and 1 ft-3 in. at EL 45 ft-6 in.). This is very small compared to the radial and circumferential dimensions. Likewise, the changes in thickness are very small compared to the radial and circumferential deformations, which are confirmed with field measurements. The larger radial and circumferential deformations observed in the field can be captured by shell elements used in the model for calculating the forces and moments needed for structural evaluations.

(3) There is through-thickness reinforcement present within the lower (below grade level) portion of the CEB walls (where there are indications of ASR) that provides a level of restraint through the thickness of the wall. The typical measured in-plane strain as determined by CI and CCI measurements is 0.2 mm/m (0.02%), which is approximately one-tenth of the yield strain of the steel reinforcement, which translates to low additional stress in the reinforcement. This strain is comparable to a 40°F temperature increase, which is similar to ambient

temperature rise. At the thickest part of the CEB wall near the base, this expansion translates to a net section growth of 0.007 in., which is very small.

(4) The through-thickness direction is not restrained except at the base, where no distress in the form of cracking or spalling has been observed, which is consistent with the very small thickness expansion discussed above.

We further note that solid elements (i.e., three-dimensional solid elements) are used to model the ring foundation, where they are more appropriate for capturing the response of thick members.

Q108. Dr. Saouma cites a discussion in the Rev. 0 CEB Evaluation (INT015) regarding the CEB foundation; Dr. Saouma appears to assert that "bubble" expansion will occur on the CEB base mat. *See* Saouma Testimony at 26 (item 10), 27-28 (fig. 16a) (INT001-R). Do you understand Dr. Saouma's criticism?

A108. (SB, GB) No, his criticism is not entirely clear to us. However, the type of "bubble" expansion postulated by Dr. Saouma is simply not possible in the CEB because the foundation is a *ring*, not a mat. As illustrated in Figure 12 below, the CEB foundation is sandwiched between the slab foundation of the Containment Building (on the interior) and concrete fill (on the exterior). Simply put, Dr. Saouma's assertion (and his figure 16a) are not applicable to the CEB.

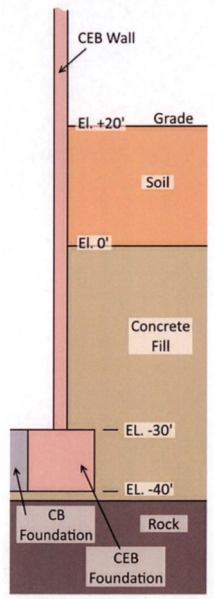


Figure 12 - CEB Foundation Diagram

Q109. Dr. Saouma makes the broad assertion that "crack index is a most unreliable indicator of ASR, and no serious researcher would rely on it." Saouma Testimony at 26 (item 11) (INT001-R). Do you agree?

A109. (MS, SB) No. The FHWA Guideline, which Dr. Saouma describes as a "reputable report," notes that an "in-situ investigation program which includes monitoring of expansion and deformation generally provides the most reliable 'prognostic' for ASR-affected structural members." FWHA Guideline at 4. As noted in A65, the use of crack indexing is consistent with ASR best practices described in various guidance documents, with the ISE Guideline (NER012) and Somerville textbook noting that CI can serve as an indicator of expansion. Similarly, the CSA Guideline notes that "monitoring the current deformations and movements is the only accurate method of determining expansion rates, and should therefore be considered for any important structure under investigation." CSA Guideline at 32 (§7.2.2) (NRC076). The CSA Guideline further notes that other factors can affect the surface width of cracks and, as described in A65, the petrographic examination of cores (including Damage Rating Index testing as recommended by Dr. Saouma in his testimony) showed that the ASR-related cracking near the surface of the concrete is similar or larger than deeper in the concrete,

indicating that the CI is a conservative indicator of ASR expansion. *See* CSA Guideline (NRC076); *see also* DRI & ASR (NER028).

Q110. Another criticism raised by Dr. Saouma pertains to the use of thermal expansion to simulate ASR expansion. *See* Saouma Testimony at 26 (item 12) (INT001-R). Is he correct that this method "will fail to capture the anisotropic nature of the expansion?"

A110. (SB) No. Structural engineers regularly use thermal expansion—a well-known and widely-accepted modeling technique—to simulate 2-D expansion of concrete members for purposes of structural evaluation. Dr. Saouma asserts that ASR expansion is volumetric (not 2-D) and will generally be larger in the through-thickness direction than in the in-plane directions ("anisotropic" means different properties in different directions). However, as mentioned in A73, the purpose of the FEM is to capture the *load effects* of ASR expansion, not to predict the dimensional level of expansion.

Thermal expansion is used as a proxy to simulate different ASR expansion in the two in-plane orthogonal directions. As discussed in A107, through-thickness deformation in the CEB—which is a thin shell structure—is very small compared to field-observed radial and tangential deformation. Therefore, the CEB modeling and evaluation are entirely appropriate. Furthermore, the CEB shell does not have any external restraint except at the foundation level, where no sign of distress has been observed in the field. And for below grade areas where there *is* through-thickness reinforcement, the measured ASR expansions are very small and do not impact the response or evaluation of the CEB. In other words, ASR expansion in the through-thickness direction is immaterial to this aspect of the evaluation, and the use of thermal expansion as a proxy for simulating the in-plane expansion is entirely appropriate.

Q111. On a related note, Dr. Saouma claims that "steel membrane elements" are included "only for the ASR study and not for the other load cases," and suggests this is improper. *See* Saouma Testimony at 26 (item 12) (INT001-R). Does he raise a valid concern?

A111. (SB) No. As stated in the material Dr. Saouma quoted from the Rev. 0 CEB Evaluation (INT015), steel membrane elements are "included in the model when applying ASR expansion and concrete swelling strains." That is because, in the mechanisms of ASR expansion and swelling, the concrete and reinforcing steel strain differently; thus, it is necessary to model them separately to determine the internal forces and stresses due to ASR. For all other types of loading, the concrete and reinforcing steel can be modeled together as a composite assembly. This is common, accepted practice which is supported by ACI 318-71. *See* ACI 318-71 ¶ 8.5.3.1 (NRC049).

Q112. Likewise, Dr. Saouma asserts that "the kinetics of the reaction is not captured, and no future predictions could be made." *See* Saouma Testimony at 26 (item 12) (INT001-R). Is this criticism valid?

A112. (SB) No. Again, Dr. Saouma seems to misunderstand the fundamental purpose of the SEM; its objective is not *prediction* of future ASR expansion. Rather, NextEra's methodology promotes the objective *monitoring* of future ASR expansion against a calculated threshold limit. The purpose of the FEM is to quantify the load effects of ASR on the structural elements at the current state of the structures and to calculate the remaining margin (i.e., the threshold limit) to the acceptance criteria in the codes of record.

Furthermore, attempting to model the kinetics of the ASR mechanism (constitutive modeling) at the macro-scale as proposed by Dr. Saouma would require having significant data collected since the time of construction, as discussed in A84 and A86. Without the (unavailable) point-by-point historical climatological data throughout every component at every structure at Seabrook, any constitutive modeling would result in an unrealistically uniform degree of ASR

throughout the structure, which would not have captured the varying levels of ASR reaction directly measured by the field observations. And the use of such results as inputs for the evaluations would have resulted in non-representative modeling of the resulting forces.

Q113. Turning to another topic, Dr. Saouma points to a statement in the Rev. 0 CEB Evaluation (INT015) citing certain research as providing the basis for assuming concrete can be expected to swell between 0.01% and 0.02%; he then claims that "this is completely arbitrary" and that "[t]here is no scientific basis for such range of values." *See* Saouma Testimony at 26-27 (item 13) (INT001-R). Is he correct?

A113. (MS, SB) No. Where the lower portion of the CEB has been exposed to water in the past, there is potential for moisture-related swelling in addition to ASR expansion; this needs to be accounted for in the analysis. The moisture-related swelling, where it occurs, is modeled by adding a concrete expansion of 0.01% in addition to the ASR expansion.

Research has shown that unreinforced concrete made of portland cement submerged in freshwater or limewater exhibits swelling that increases to about 0.02% after 30 years. *See* A.M. Neville, PROPERTIES OF CONCRETE, 5th ed. (2012) (NER039); J.J. Brooks, *Accuracy of Estimating Long-Term Strains in Concrete*, 36 MAGAZINE OF CONCRETE RESEARCH 127, 131-45 (June 1984) (NER040); J. Mijnsbergen & H. W. Reinhardt, *Long-Time Creep and Expansion Behavior of Concrete in a Marine Environment*" ACI SP 109-27 (1994) (NER041); G.E. Troxell, et al., *Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete*," 1958 PROCEEDING OF THE AMERICAN SOCIETY FOR TESTING AND MATERIALS 1101-20 ("Troxell") (NER042).

Researchers also report that the creep, shrinkage, and swelling of reinforced concrete are less than the unreinforced concrete samples. Troxell reported that the effect of reinforcement reduces the creep stresses by about 55%. Using a similar reduction of recommended swelling after 30 years leads to a swelling factor of 0.01%. *See* Troxell (NER042) Accordingly, the CEB

Evaluation conservatively used this value to represent the swelling behavior of buried segments of the CEB structure. This is a conservative assumption, since only a very limited area at the lower depth of the CEB was exposed to standing water, and the duration of the standing water has been much less than 30 years. Thus, contrary to Dr. Saouma's assertion, these values are far from arbitrary, and are based on published scientific research.

Q114. Next, Dr. Saouma raises a concern about seismic modeling. More specifically, he notes that the Rev. 0 CEB Evaluation employed a response spectra analysis using a simplified "stick" model, and then argues that "the stick model is a model of the past," is "overly simplistic," and "cannot capture the seismic contact between the wall and the adjacent soil" absent certain adjustments. *See* Saouma Testimony at 27 (item 14) (INT001-R). Do his comments offer a valid criticism of NextEra's seismic modeling?

A114. (SB) No. The CEB seismic calculation involves: (1) calculation of seismic responses using a lumped-mass stick model using the response spectra analysis method for calculating acceleration; and (2) use of calculated accelerations as inertial loads for a detailed finite element model for calculating seismic forces to be used for structural evaluation. In other words, the Rev. 0 CEB Evaluation uses what is referred to as the "multi-step method." Dr. Saouma's concern about using a "simplified 'stick' model" for calculating seismic forces in the CEB appears to overlook the second part of the methodology.

In any event, lumped-mass stick modeling for calculating seismic responses is a valid and accepted method. It has been endorsed by the NRC in SRP § 3.7.2 at 11 (Rev. 4, Sept. 2013) (NER043),[11] as well as in ASCE 4-16 § 3.1.2.1(a) (NER036). ASCE 4-16 § 3.1.2(b) (NER036) also endorses the "multi-step method" (i.e., applying seismic accelerations calculated from the lumped-mass stick model, Step 1, to a more detailed model, Step 2, for calculating structural forces for evaluation). The nodal acceleration calculated at Step 1 for the CEB is applied to the

[11] NER043 is an excerpt of Section 3.7.2 from NUREG-0800.

detailed finite element model of the CEB as Step 2 to calculate seismic forces and moments to be used for structural evaluation. The lumped-mass stick model was developed and analyzed as part of Seabrook's original design process and has been described in the UFSAR for calculating overall seismic responses using a response spectrum analysis method that is endorsed by the NRC in SRP § 3.7.2 at 1 (NER043).

The lumped-mass stick model has also been used and approved by the NRC for seismic analysis of many seismic Category I nuclear plant structures, including recent approval of the containment, turbine, and control buildings of the South Texas Plant Units 3 and 4 combined operating license. The stick models for the turbine and control buildings used in these analyses are comparable to or simpler than the CEB seismic lumped-mass stick models. The dynamic soil pressure from soil adjacent to the CEB is also considered in the analysis, and evaluation based on the pressure profiles is described in UFSAR § 3.7(B).2.

Q115. Dr. Saouma also claims that the "[i]mpact of soil through proper deconvolution or soil structure interaction is not accounted for as required by ASCE 4-16 (2016)." *See* Saouma Testimony at 29 (item 18 & fig. 17) (INT001-R). Is this an accurate statement?

A115. (SB) No. Briefly stated, Dr. Saouma asserts that NextEra should have used soil-structure interaction ("SSI") analysis instead of "fixed base" analysis in the Rev. 0 CEB evaluation. (In fixed base analysis, the design seismic motion is applied at the fixed base of the structure, not through the soil below or surrounding the structure's base, as in SSI). However, the hard rock on which the Seabrook structures bear is stiff enough that soil-structure effects are not significant. More specifically, the Seabrook structures are founded on hard rock with shear wave velocity of 8,000 to 10,000 ft/sec. *See* UFSAR Section 2.5.2.5 (NER044). ASCE 4-16 and the SRP specify that fixed base analysis is appropriate (and no SSI analysis is necessary) when the shear wave velocity is greater than 8,000 ft/sec. *See* ACI 4-16 § 5.1.1(a)(3) (NER036); SRP

§ 3.7.2 (NER043). Therefore, because Seabrook's shear wave velocity is greater than 8,000 ft/sec, fixed base seismic analysis is entirely appropriate for the CEB Evaluation.

Q116. Moving to Item 15, Dr. Saouma notes the "section cut approach" used in the Rev. 0 CEB Evaluation and claims that this approach reduces the assessment to a "series of parallel column[s] with no interaction among them." ***See*** **Saouma Testimony at 28 (item 15) (INT001-R). Is he correct?**

A116. (SB) No. Dr. Saouma misunderstands the section cut approach, which is commonly used in structural engineering to design and evaluate structural elements. The section cut approach is a post-processing approach that uses the results from the finite element analysis model, with all of its connectivity, properties, loading, and boundary conditions. The output from a series of consecutive elements is aggregated to a load over a length (so called "section cut") and compared to the capacity across the length of the section cut.

To illustrate this, Figure 13 below shows three representative section cuts used in the Rev. 0 CEB Evaluation. This figure shows that the section cuts are simply selections from the finite element model, not a series of isolated clusters of elements. The section cuts are defined to capture the peak demand in a particular portion of the structure. The section cut approach captures the actual structural response and evaluation in line with code equations and the supporting structural testing.

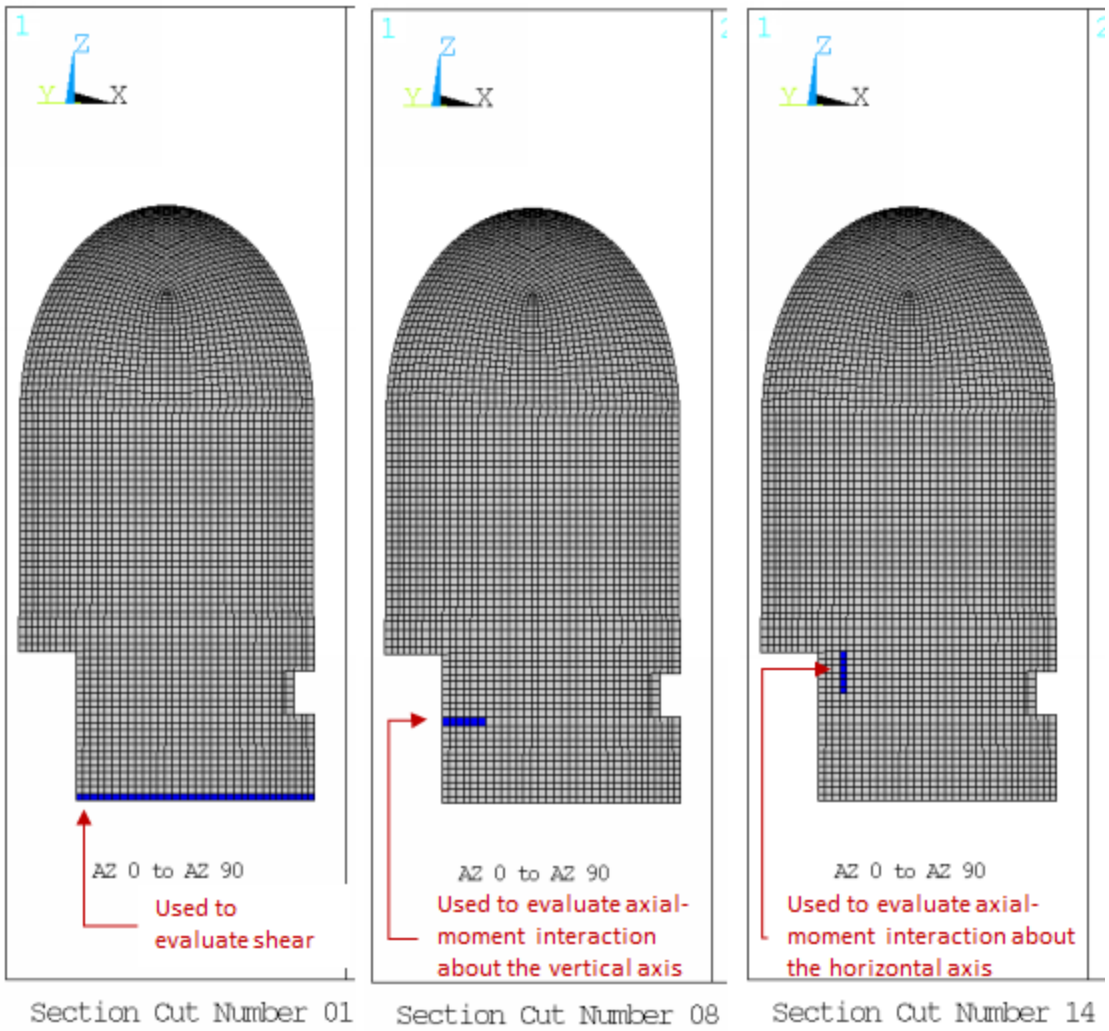


Figure 13 - Three representative section cuts used in the Rev. 0 CEB evaluation

Q117. Is sectional (or section cut) analysis consistent with code-based design and evaluation?

A117. (SB) Yes. Section 9.1.1 of ACI 318-71 states "Structures and structural members shall be designed to have strengths at all *sections* at least equal to the structural effects of the design loads and forces in such combinations as are stipulated in this Code." ACI 318-71 § 9.1.1 (NRC049). ACI 318-71, Section 11.16.1, which provides special shear provisions for walls, uses a section equal to 80% of the wall length to evaluate in-plane shear. *See id.* at § 11.6.1. Additionally, Section 11.16.6 refers to Section 11.10 for out-of-plane shear provisions in walls and slabs. *See id.* at § 11.6.6. Section 11.10.1(a) specifies evaluation for one-way shear

considering the member (wall, slab, or footing) acting "essentially as a wide beam, with a potential diagonal crack extending across the entire width." *See id.* at § 11.10.1. Section 11.15.2, which relates to shear-friction, states "[a] crack shall be assumed to occur along the shear path." *Id.* at § 11.15.2. Clearly, a sectional analysis approach is supported by the Code. The ACI Code also recommends that walls and slabs designed using distributed reinforcement (as is used for seismic Category I structures at Seabrook) have sufficient ductility to redistribute peak demands that may be calculated by the finite element method or due to concentrated loads.

Q118. Relatedly, Dr. Saouma criticizes this approach and claims that NextEra "should have examined indeed element by element and assess strength through established failure criteria." *See* Saouma Testimony at 28 (item 15) (INT001-R). Is he correct?

A118. (SB) No. As explained in response to previous questions, the SEM and corresponding acceptance criteria flow from the original design basis and all of its referenced codes and standards. The ACI Code employs sectional analysis and design, supported by decades of application and testing, and uses well-established acceptance criteria. Dr. Saouma provides no basis for his assertion that the SEM should have abandoned this robust code-based approach in favor of using element-by-element analysis coupled with *theoretical* acceptance criteria.

Q119. Additionally, Dr. Saouma claims that "serviceability" can only be quantified through a nonlinear analysis. *See* Saouma Testimony at 28 (item 15) (INT001-R). Is this a valid criticism of NextEra's SEM?

A119. (SB, GB). No. "Serviceability" relates to the performance of the structure under normally-expected service loadings (as opposed to maximum loads that occur due to extreme events such as safe-shutdown earthquake or factored load combinations). As shown in Figures 9 and 10, the serviceability loads are much lower than the factored design loads; accordingly, at

service load levels, the structures will be well within the linear and elastic range. Therefore, linear analysis is sufficient (as discussed in A94and A95) for evaluation of service load behavior. Furthermore, deformation serviceability is monitored via the SMP to make sure the seismic gaps between structures remain above defined gap requirements, and impacts on any SSC within each structure (or between structures) are monitored based on deformation limits.

Q120. Dr. Saouma discusses an assumption from Appendix F of the Rev. 0 CEB Evaluation. *See* Saouma Testimony at 28 (item 17) (INT001-R). Do you understand his criticisms?

A120. (SB) Not entirely. As best we can tell, he (1) appears to repeat his argument in Section C.2.2.2 of his Testimony that "the FSEL tests modeled only the out of plane shear and not the in-plane," and (2) appears to believe that the Rev. 0 CEB Evaluation "implies that cracks will only occur along the direction of seismic excitation." Saouma Testimony at 12, 28 (INT001-R).

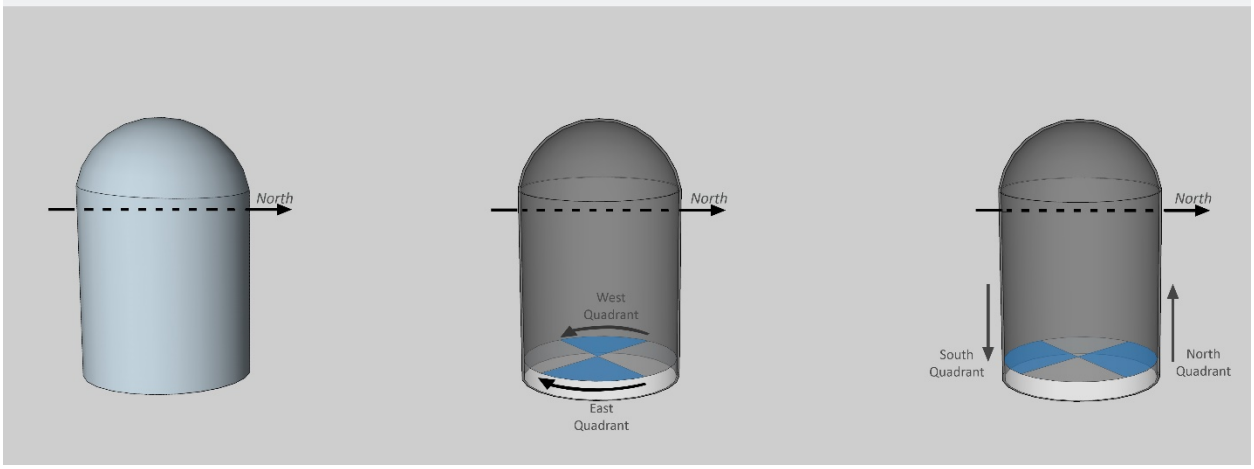
Q121. How do you respond to the first part of his criticism regarding shear?

A121. (SB) First, the MPR Testimony (NER001) at A202 and A203 addresses Dr. Saouma's criticism of the LSTP. Second, to the extent his comment somehow applies to the Rev. 0 CEB Evaluation, we note that the seismic responses of the CEB are calculated using the *multi-step method* discussed in A114, where the accelerations calculated from the lumped-mass stick model are applied to the detailed finite element model of CEB. The in-plane shear, out-of-plane shear, axial, and moment responses are calculated using the detailed finite element model of the CEB. As discussed in A116, these responses are calculated both on the element level and the section cut level. Thus, the in-plane shear and out-of-plane shear are not confused or transferred.

Q122. How do you respond to the second part of Dr. Saouma's criticism, regarding cracking direction?

A122. (SB) The SEM details the procedure for when structural cracks can be initiated and how to calculate the cracked section properties for all types of cracks: flexure, axial, and shear (in-plane and out-of-plane). But contrary to Dr. Saouma's assertion, neither the SEM nor the Rev. 0 CEB Evaluation imply that cracking can *only* happen in the direction of seismic motion. As explained further below, in-plane shear cracking due to seismic excitation is not *expected* in the CEB due to the very low seismic motion at Seabrook.

By way of background, the CEB structure consists of a spherical dome on a cylinder (with large penetrations) that is supported by a ring foundation. Consider the simplified example in Figure 14, which shows a cylinder without penetrations. Under north-south seismic excitation, the east and west quadrants provide *shear* stiffness, and the north and south quadrants provide *overturning* stiffness. Figure 14 schematically shows this behavior, where Figure 14a illustrates seismic demand toward the north, Figure 14b shows the northward seismic demand resisted by in-plane shear on the east and west quadrants at the base, and Figure 14c shows the seismic overturning resisted by vertical forces at the base in the north and south quadrants. This behavior can be captured by the multi-step method discussed in A114. (Because there are large penetrations in the CEB cylindrical shell, the finite element analysis correctly redistributes the loads that could result in reactions different than the simplified example shown in Figure 14).



(a) Seismic Inertia (b) Base Shear (c) Overturning

Figure 14 - Seismic load transfer to foundation

Additionally, in-plane shear cracking due to seismic excitation is not expected in the CEB due to the *very low seismic motion* at Seabrook—equivalent to 12% of gravity for a rigid structure. (In technical terms, the zero period acceleration for the OBE is 0.12g). Moreover, the seismic response of the CEB is insensitive to assumptions regarding cracking. As shown in Figure 15 below, the lateral fundamental frequency of the CEB (if uncracked) is about 4.2 Hz (the solid vertical line). If we *conservatively* assumed the lateral shear stiffness of the structure was reduced by 50% due to in-plane shear cracking, it would only cause minor changes (less than 5%) in acceleration used for the seismic response calculation due to the shift of the lateral fundamental frequency (the dashed vertical line in Figure 15). This shift is not material because the lateral fundamental frequency of the CEB (if uncracked) is *already* on the top plateau of the design response spectra (the solid blue line in Figure 15 below).

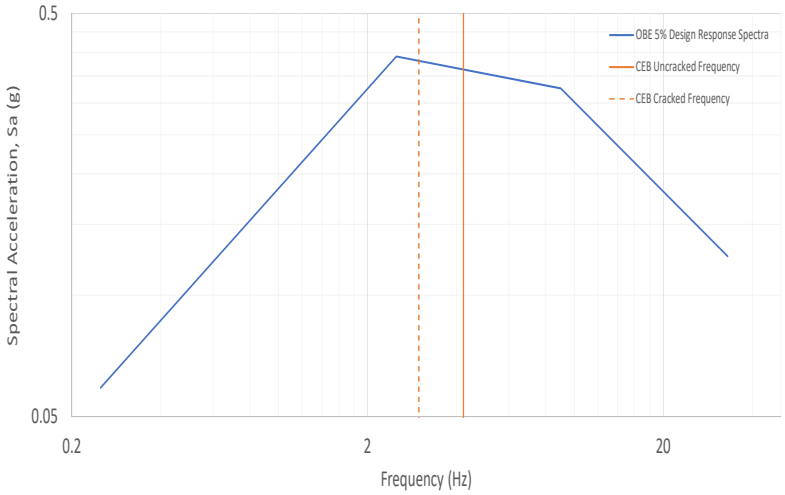


Figure 15 - OBE 5% damped design spectra, fundamental lateral frequency of CEB

In summary, in-plane cracking is unlikely due to low seismic motion at Seabrook; and even if in-plane cracking did occur, it would not materially impact the seismic response. Thus, Dr. Saouma's comment does not identify a deficiency in the methodology.

VI. CONCLUSION

Q123. Please summarize your testimony and the bases for your conclusion regarding the admitted contention.

A123. (MS, SB, GB) The Contention lacks merit because the SEM provides a methodology for structural evaluations of seismic Category I structures at Seabrook that fully complies with all applicable legal and regulatory requirements. Our testimony describes the key elements of the SEM and the reasons why the SEM approach is acceptable.

The criticisms provided by Dr. Saouma are based largely on his preference for using a constitutive modeling approach to analyze ASR. However, Dr. Saouma does not show that usage of a methodology *other* than his preferred method is inherently unacceptable. As we have explained, Dr. Saouma's approach is to *predict* extreme levels of ASR that are well beyond what is expected at Seabrook; whereas, the approach NextEra selected was to establish the bounds within which the code-based UFSAR equations and procedures for calculating concrete strength

remain valid (i.e., the LSTP); *monitor* Seabrook's concrete to ensure it remains within those bounds (i.e., the SMP); and analyze the structures accordingly (i.e., the SEM). This general approach has long been used by nuclear power plant licensees to manage other aging effects. And Dr. Saouma fails to explain how it is inadequate.

Furthermore, Dr. Saouma's critique of the Rev. 0 CEB Evaluation—which was not a document approved (or requested to be approved) by the NRC in the LAR—and which, in any event, has been superseded—identifies no deficiency in the LAR methodology. Overall, Dr. Saouma's claims lack merit; and C-10 does not prove the assertions in the original Contention.

Q124. Does this conclude your testimony?

A124. (MS, SB, GB) Yes.

Q125. In accordance with 28 U.S.C. Section 1746, do you state under penalty of perjury that the foregoing testimony is true and correct?

A125. (MS, SB, GB) Yes.

Respectfully Submitted,

Executed in accord with 10 C.F.R. § 2.304(d)
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